

Evaluation of 1986 AASHTO Design Guide for Jointed Concrete Pavements

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The new AASHTO Guide for Design of Pavement Structures (1986) is evaluated using theory and field performance data. Jointed plain (JPCP) and jointed reinforced (JRCP) concrete pavements are considered. The conceptual evaluation considers the assumptions, capabilities, and theoretical limitations of the AASHTO Guide. The analytical field data evaluation includes two approaches: predicted vs. actual ESALs for in-service pavements from NCHRP Project 1-19, and the specific design evaluation. Four broad zones in the United States were used to characterize the performance of the AASHTO designs in different climates. This dual approach to design procedure evaluation provides an overall picture of the capabilities of the new AASHTO Guide. Deficiencies are identified that must be considered by agencies implementing the new AASHTO Guide, and recommendations on how to overcome these deficiencies are provided.

The AASHTO structural slab thickness design model was originally developed using the results from the AASHTO road test conducted from 1958 to 1960 near Ottawa, Illinois. The AASHTO Interim Guide for Design of Rigid Pavement Structures (1) was first developed in 1962 and revised in 1972 (2) and 1981 (3). During 1984-85 the Subcommittee on Pavement Design and a team of consultants revised the existing guide under NCHRP Project 20-7/24 and issued the current version (4). A complete description of the development of the original structural design model is given in the Appendix of the 1981 Interim Guide (3) and in the AASHTO Road Test Report (5).

Since the AASHTO Design Guide is widely used for the design of rigid highway pavements, it is important to know its capabilities and limitations and the performance of the AASHTO pavement designs under various conditions. The data base used in this study was developed under NCHRP Project 1-19 (6).

The AASHTO Guide is evaluated conceptually and analytically.

CONCEPTUAL EVALUATION

The conceptual evaluation includes a review of the Guide's fundamental basis for development and a summary of the conceptual weaknesses and limitations of the design procedure.

Fundamental Basis for Development

Performance data from the AASHTO Road Test and regression techniques were used to develop the following empirical model:

$$\log_{10} W = \log r + G/B \quad (1)$$

where

W = axle load applications, for load magnitude $L1$ and axle type $L2$, to a serviceability index of $P2$,

$$\log_{10} r = 5.85 + 7.35 \log (D + 1) + 4.62 \log (L1 + L2) + 3.28 \log (L2),$$

$$B = 1.0 + [3.63(L1 + L2)^{5.20}]/[(D + 1)^{8.46}L2^{3.52}],$$

$$G = \log [(P1 - P2)/(P1 - 1.5)],$$

D = PCC slab thickness (in.),

$L1$ = load on a single or a tandem axle (kips),

$L2$ = axle code, 1 for single axles and 2 for tandem axles,

$P1$ = initial serviceability index, and

$P2$ = terminal serviceability index.

This empirical model (Eq. 1) is applicable only to the Road Test climate, pavement materials, subgrade, and slab design. To expand its applicability, it was modified by the inclusion of Spangler's corner stress equation, which incorporates slab thickness, load transfer coefficient, material properties such as portland cement concrete (PCC) flexural strength (F), modulus of elasticity (E), and foundation support (k). Basic assumptions made in this extension are:

- There will be no variation in W for different load magnitudes if the level of the ratio of tensile stress/strength of the PCC slab is kept constant and such W will be accounted for by AASHTO Road Test Equation 1.

- Any change in the ratio of tensile stress/strength resulting from changes in the values of E , k , and F (modulus of rupture) will have the same effect on W as an equivalent change in slab thickness (calculated by Spangler's equation) will have on W (as per Equation 1).

In the 1986 AASHTO Guide, more empirical factors are included to adjust the designs for various conditions.

1. Design reliability concepts are introduced with the objective of decreasing the risk of premature structural deterioration below acceptable levels of serviceability. The reliability design factor (F_R) accounts for chance variations in both the traffic prediction and the pavement performance prediction for a given W_{18} . This reliability design factor provides a pre-

determined level of reliability ($R\%$) that pavement sections will survive the traffic for which they were designed. It applies only to structural deterioration, however, not to loss of serviceability due to nonload causes (e.g., joint deterioration, swelling soils).

2. The joint load transfer factor, J , is extended to consider different types of shoulders. A value of 3.2 is recommended for the J factor for a jointed plain (JPCP) or jointed reinforced (JRCP) concrete pavement with a mechanical load transfer device (e.g., dowel bars) at joints in pavements with asphalt concrete shoulders, whereas the values ranging from 2.5 to 3.1 are recommended for the same pavement with tied PCC shoulders. Nondoweled pavements have a recommended J ranging from 3.8 to 4.4 for AC shoulders, and from 3.6 to 4.2 for tied PCC shoulders. This method adjusts for different load transfer conditions by varying slab thickness.

3. A drainage coefficient (C_d) multiplier is added to the design equation. Its value represents the quality of drainage and the percent of the time the pavement structure is exposed to moisture levels approaching saturation. It provides a way to approximate the effect of drainage by means of modifying the load transfer coefficient, J . The C_d value for AASHTO Road Test subdrainage conditions is 1.0.

4. The potential effect of subgrade swelling and frost heave on the rate of loss in serviceability is considered. The thaw-weakening and seasonal variations in modulus of subgrade reaction are also considered in the new Guide.

5. The subgrade soil-resilient modulus, M_R , is used in the new Guide to estimate the effective modulus of subgrade reaction, k , at the top of the subbase. A loss-of-support factor (LS) included in the design accounts for potential subbase erosion and/or differential vertical soil movement by diminishing the overall effective k value on the basis of the size of the void that may develop beneath the slab. Some suggested ranges of LS , depending on the type of subbase material, are provided. This results in a pavement design for k values as low as 25 pci.

The current structural design model is:

$$W_{18} = W_{18}/F_R = W_{18}/10^{-Z_R S_o} \quad (2)$$

where

$$\begin{aligned} \log W_{18} = & 7.35 \log (D + 1) - 0.06 \\ & + Gt/[1 + 1.624 \times 10^7/(D + 1)^{8.46}] \\ & + (4.22 - 0.32 P_i) * \log [(S'_c * C_d (D_{0.75} - 1.132) \\ & \div (215.63J) * (D_{0.75} - 18.42)/(E_c/k)_{0.25})] \end{aligned} \quad (3)$$

where

- $Gt = \log [(P_i - P_t)/(4.5 - 1.5)],$
- $W_{18} =$ mean predicted total number of 18-kip equivalent single-axle load applications in the design life,
- $F_R =$ reliability design factor,
- $Z_R =$ standard normal deviate corresponding to selected level of reliability,
- $S_o =$ overall standard deviation for rigid pavement,
- $D =$ mean thickness of pavement slab (in.),
- $P_i =$ initial serviceability index,
- $P_t =$ terminal serviceability index,

- $S'_c =$ mean modulus of rupture for PCC used on specific project,
- $J =$ load transfer coefficient used to adjust for load transfer characteristics of specific design,
- $C_d =$ drainage coefficient,
- $E_c =$ mean modulus of elasticity for PCC (psi), and
- $k =$ mean modulus of subgrade reaction (pci).

Weaknesses and Probable Limitations

Some major weaknesses and probable limitations of the procedure in designing against important distress types existing in rigid pavement are summarized below.

Accuracy of Structural Design Model

The empirical model of Equation 1 was derived from results from the Road Test data and relates to its specific Road Test conditions. Within these conditions, the ability of Equation 1 to predict the exact number of load applications to any given level of serviceability index for a pavement is shown in Figure 1 (7). The shaded band indicates the range in load applications that includes approximately 90 percent of all the performance data. In the top curve of Figure 1, for example, for slab thicknesses of 8 inches, the resulting number of 30-kip single-axle applications to a terminal serviceability index of 2.0 ranged between 400,000 and 1,910,000 for controlled AASHTO Road Test conditions. If Equation 1 is used for conditions other than those for which it was developed, its range of accuracy or associated error of prediction will be greater. This may be particularly true for different climatic conditions. The modified expression, Equation 3, allows for changes in material properties (S'_c , E_c , and k), but the accuracy of these adjustments is unknown.

Variability

A serious limitation of the AASHTO design procedure is that the empirical design model (Equation 1 or 3) is based on very short pavement sections where construction and material quality were highly controlled. Typical highway projects, which are normally several miles long, contain much greater construction and material variability and hence show more variability in performance along the project in the form of localized failures. Projects designed using the Guide would, therefore, tend to show significant localized failures before the average project serviceability index (PSI) drops to P_t , unless a level of reliability higher than 50 percent was selected for the design.

Design Period

Design periods under consideration usually range from 20 to 40 years. The number of years and 1.1 million applications, upon which Equation 1 is based, represent only a fraction of the load applications that would be expected on high-volume pavements over the design period (10 million to 100 million

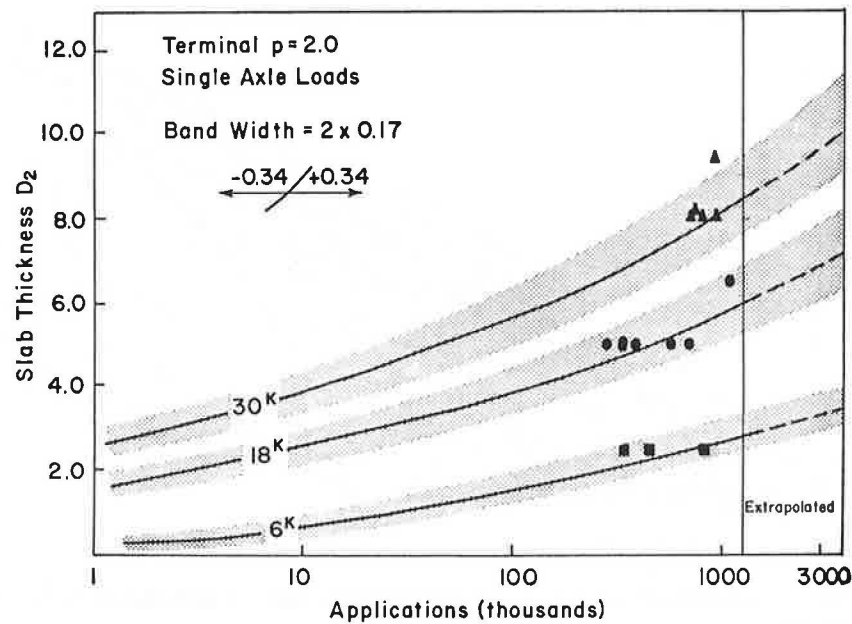


FIGURE 1 Illustration of error of prediction of basic AASHTO design model (7). Shaded bands indicate range in load applications that includes approximately 90 percent of all the performance data.

applications). Even if these equations can be extrapolated for the large difference in the number of load applications, there are several climatic effects that occur over time (as represented by age) which cause severe deterioration of the pavements even without heavy load applications (e.g., corrosion of steel, joint freezeup, D-cracking, reactive aggregate, incompressible buildup in joints). Therefore, in similar or more severe climates, the pavements would be expected to endure fewer load applications and fewer years than predicted by Equation 1 or 3. In mild climates, pavements may perform much better than predicted.

Loss of Support

The Road Test used a specific set of pavement materials and one roadbed soil. Many of the Road Test sections showed severe pumping of the subbase with loss of support. Therefore, Equations 1 and 3 are biased toward this high loss-of-support condition.

Joint Design

Only one type of joint design was used in the AASHTO Road Test. If other types are used, such as joints without dowels or with some unusual type of load transfer devices, the pavement life would be significantly changed. The type of base would also affect load transfer and, thus, performance. Basic deficiencies in the joint design recommendations include little or no guidance on joint spacing, on rational determination of dowel size and spacing, on when mechanical load transfer devices are required, and on load transfer systems other than dowels.

Drainage Design

In the new Guide, drainage effects are considered in terms of the effect of moisture on subgrade strength and on base erodability. For new rigid pavement design, the effect of drainage is considered by modifying the load transfer coefficient, J , through the C_d factor. It is well known that the subdrainage conditions at the Road Test were very poor because of the huge amount of pumping. Despite this, AASHTO recommendations for C_d show a value of 1.0 to correspond to a quality of drainage of good to fair for subgrade saturation levels of greater than 5 percent, which surely existed at the Road Test. This recommendation cannot be adjusted properly for poor drainage conditions. Another question is whether poor subdrainage can be considered by simply increasing slab thickness. This is not likely since a thickness increase only decreases deflection slightly.

Reinforcement Design

Slab reinforcement is designed using the subgrade drag theory. The mathematical expression for subgrade drag theory used for longitudinal reinforcement design is a major simplification of the actual forces encountered. The most significant limitation arises if the unrestrained slab length (i.e., distance between joints) assumed in reinforcement design is altered through a partial or complete seizing of one or more joints. This phenomenon could cause a significant increase (double or more) in the steel stress, which may result in yielding or rupture of reinforcement at an intermediate crack between joints. Also, the procedure does not allow for loss of effective reinforcement through corrosion. It is expected, therefore, that long joint spacings in cold climates, accom-

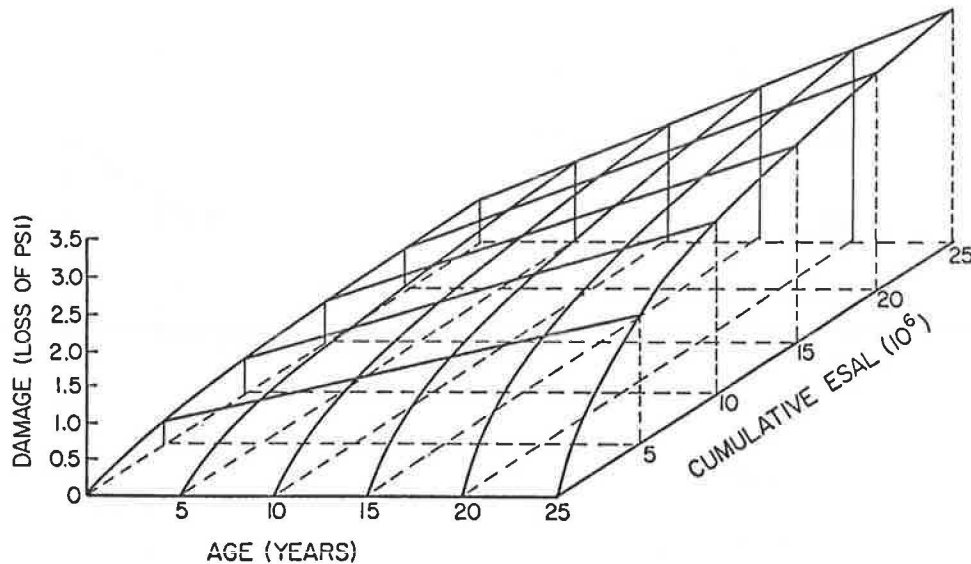


FIGURE 2 Sensitivity of Illinois damage (serviceability) model to cumulative load repetitions and age (6).

panied by joint seizure, would result in rupture of the reinforcement with subsequent faulting and spalling of cracks.

Climate

Concrete pavement performance is highly dependent on the climatic conditions, and evidence exists that climatic conditions could have a significant effect on pavement life (8). Since the AASHO Road Test was conducted over a period of only two years, climatic effects were not as significant as if the same traffic had been applied over a longer period of, say, 20 to 40 years. Steel corrosion requires several years to develop into a serious condition, so joint lockup and subsequent yielding of the steel reinforcement for JRCP pavements would logically not occur for at least several years after initial construction. Figure 2 shows the results of a life prediction model developed from the Illinois data base (6), where age and traffic data were available over short as well as long time periods. An interaction between age and traffic can be observed in that there is much greater pavement damage from heavy traffic over long periods than if the same traffic was applied over a shorter time period.

ANALYTICAL EVALUATION

Predicted Versus Actual ESALs

The actual number of ESALs was compared to the predicted ESAL due to the measured loss in present serviceability index, using the original AASHO performance equation (i.e., Eq. 3). This comparison was made for each section of JPCP and JRCP in the data base. The actual pavement thicknesses, material properties, serviceability at the time of the study, and actual traffic were input into the equation. The drainage coefficient value for the equation was set at 1.0. The value of the J factor was 3.2 for joints with dowels and 4.1 for joints

with aggregate interlock (without dowels). The data retrieval and computations were completed by utilizing the Statistical Package for the Social Sciences (9). The analysis was run at the 50 percent level of reliability.

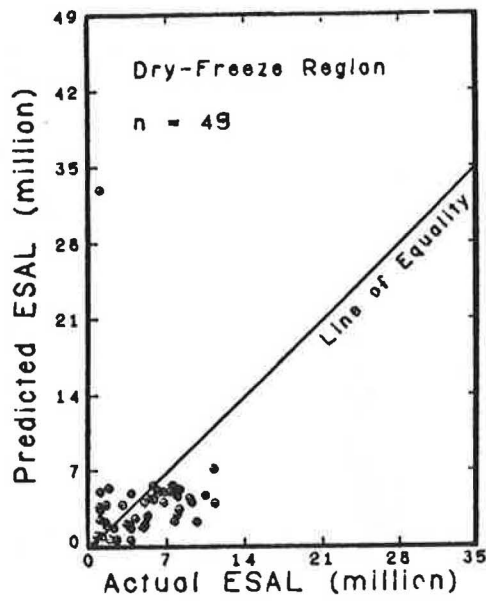
The pavement sections in the data base were divided into four broad climatic zones and the results were compared by zone. The data base contains jointed pavement sections from the following states: California (wet-freeze JPCP), Georgia (wet-nonfreeze JPCP), Illinois (wet-freeze JPCP and JRCP), Louisiana (wet-nonfreeze JRCP and JPCP), Minnesota (dry-freeze and wet-freeze JRCP), Nebraska (dry-freeze JRCP), and Utah (dry-freeze JPCP). The plots of predicted vs. actual ESALs for each climatic zone for JPCP and JRCP are given in Figure 3 and Figure 4, respectively.

A summary of the results of the predicted vs. actual ESALs for JPCP and JRCP is given in Table 1. The significance and comparison of the results are discussed below.

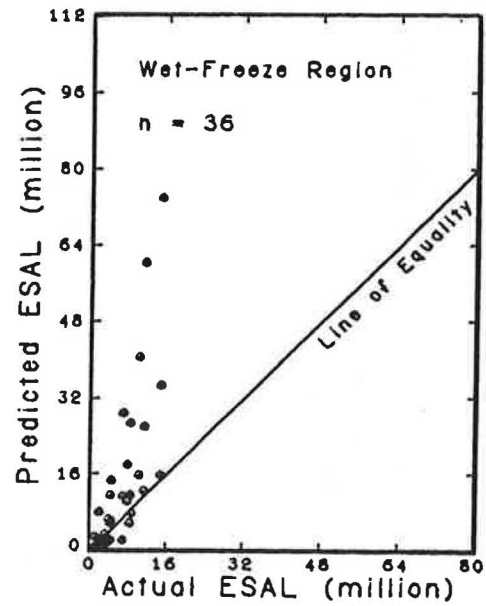
JPCP

The results are highly dependent on climate. Almost all the sections in the dry-nonfreeze climate performed better than the original AASHO model predicted (52 of 53 sections, or 98 percent acceptable). None of these pavements contained dowels as did the AASHO Road Test sections. The sections in the dry-freeze and wet-nonfreeze climates performed generally as predicted with 60 and 71 percent of the sections acceptable, respectively.

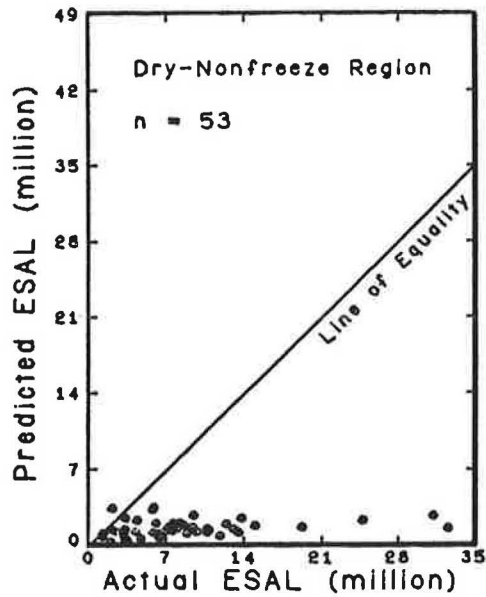
The JPCP sections in the wet-freeze climate (same as AASHO Road Test) performed worse than the AASHO model predicted with only 9 of 36 sections (or 25 percent) acceptable. The average actual ESALs in wet-freeze climate are 7.36 million less than the predicted ESALs (or 92 percent of the average actual ESALs). The JPCP sections performed adequately in the wet-nonfreeze and dry-freeze zones, with the actual ESALs approximately equal to the predicted ESALs.



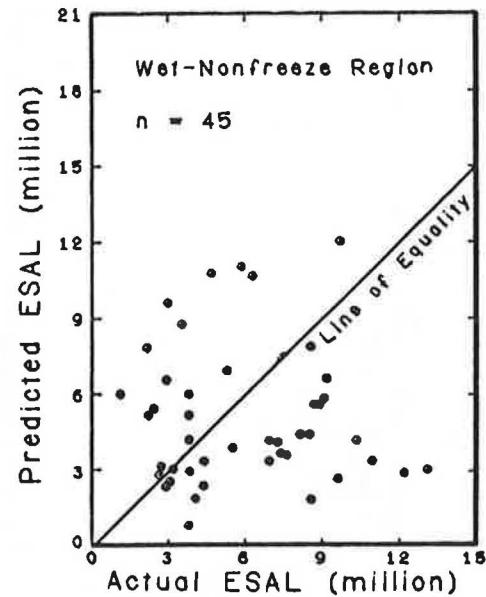
(a)



(b)



(c)



(d)

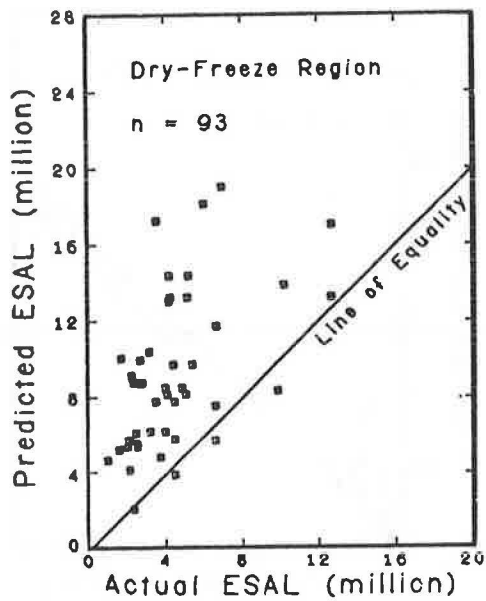
FIGURE 3 Predicted ESALs vs. actual ESALs for JPCP, using original AASHO Road Test PSI prediction model.

Figure 3(d) shows that a higher variation exists in prediction in the wet-nonfreeze zone. The results shown in the dry-freeze zone, that 86.9 percent of the average actual ESALs are less than the predicted ESALs, are somewhat biased because one slight-trafficked 13-in. slab section has an erratically high loss in PSI. See Figure 3(a).

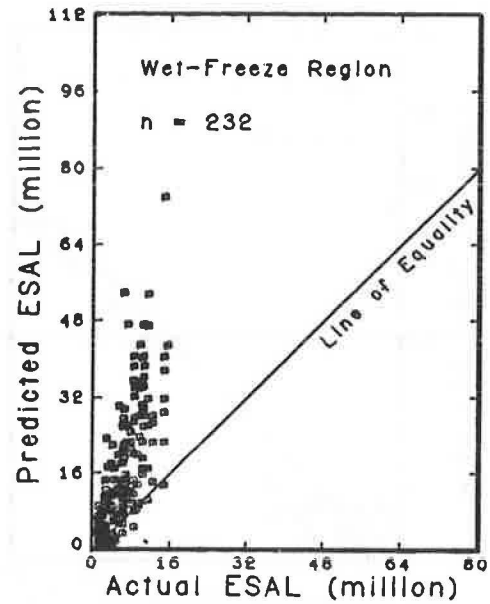
The original AASHO equation seems to provide overly conservative JPCP structural designs in the dry-nonfreeze climate, adequate designs in the dry-freeze and wet-nonfreeze climates, but inadequate designs in the wet-freeze climate, all at the 50 percent reliability level.

JRCP

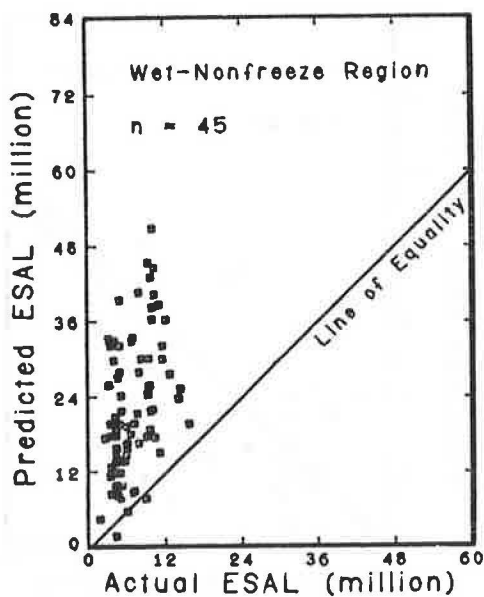
The results in Table 1 show that the JRCP sections did not perform as well in any climate as the original AASHO model predicted. For all the JRCP sections, only 41 of 374 (or 11 percent) performed better than predicted. The average actual ESALs for all JRCP sections were 10.58 million less than the predicted ESALs (or 193.6 percent of the average actual ESALs). On the basis of these results, the original AASHO equation does not provide adequate JRCP structural designs for any climate at the 50 percent reliability level.



(a)



(b)



(c)

FIGURE 4 Predicted ESALs vs. actual ESALs for JRCP, using original AASHO Road Test PSI prediction model.

Climatic factors affect JRCP more severely than JPCP. Many of the JRCP sections in the NCHRP 1-19 data base have deteriorated because of factors other than traffic loading. Thus, the AASHO model does not provide adequate JRCP designs. The inherent variability in the prediction of pavement section performance indicates that there is a need to increase the reliability level and other design improvements if the equation is to provide adequate designs for JRCP sections.

Specific Design Evaluation

Results obtained from Heinrichs et al. (10) showed that a significant difference in pavement performance occurred only between the four major zones: wet-freeze, wet-nonfreeze, dry-freeze, and dry-nonfreeze. The specific design evaluation was then conducted using climatic data from the data base. The classification for the four broad climatic zones is the same

TABLE 1 SUMMARY OF RESULTS FOR ORIGINAL AASHO ROAD TEST PSI PREDICTION MODEL

Climatic Region (1)	# of Cases (2)	# of Section Acceptable* (3)	Percent Acceptable (4)=(3)/(2)	Mean Difference** (5)	Mean Percent Difference*** (6)
JPCP:					
Wet - Nonfreeze	45	27	60%	0.911	-17.1%
Wet - Freeze	36	9	25%	-7.355	-92.0%
Dry - Nonfreeze	53	52	98%	6.405	69.5%
Dry - Freeze	49	35	71%	0.763	-86.9%
Overall	183	123	67%	0.836	-25.4%
JRCP:					
Wet - Nonfreeze	93	3	3%	-15.460	-282.0%
Wet - Freeze	232	34	15%	-9.886	-168.8%
Dry - Nonfreeze	-****	-	-	-	-
Dry - Freeze	49	4	8%	-4.584	-143.0%
Overall	374	41	11%	-10.577	-193.6%

* "Acceptable" means the actual number of 18-kip ESALs is equal or greater than the predicted ESALs, i.e. the pavement section performed as good as or better than the AASHO model predicted, otherwise is "unacceptable".

** Difference = Actual ESAL - Predicted ESAL, in millions

*** Percent Difference = [(Actual ESAL - Predicted ESAL)/Actual ESAL] x 100%

**** No JRCP section is available in dry-nonfreeze region in COPES.

as used in the above predicted vs. actual ESAL evaluation. The prevailing values of the climatic data in each climatic zone as averaged from the data base are shown in Table 2.

A number of pavement design situations were developed for JPCP and JRCP over the four climatic zones. The design factors that were varied included two subgrade soils (fine-grained and coarse-grained), with and without dowels (JPCP only), and shorter and longer joint spacings (JRCP only). The fine-grained subgrade soils were defined as A-7-5 or A-7-6 in the AASHTO soil classification, and the coarse-grained subgrade soils were defined as A-2-6 or A-2-7. The drainage of these two types of subgrade soils was characterized as poor for the fine-grained soils and good for the coarse-grained soils. A resilient modulus of 3,000 psi was assumed for the fine-grained soils and 7,000 psi for the coarse-grained soils. The elastic k values for fine- and coarse-grained soils were assumed to be 100 pci and 190 pci, respectively, when the subgrade degree of saturation was compacted to 70-90 percent. The

1986 AASHTO Guide was used to generate pavement designs for JPCP and JRCP for each of the design situations. The design life was 20 years, and the design traffic was 15 million 18-kip ESALs in the design lane.

Specific soil, subbase, concrete, and other properties for the designs are shown in Table 3. The climatic design inputs for the new AASHTO Guide for each of the four climatic zones are given in Table 4. The values of the drainage coefficient C_d and loss-of-support factor LS were used as recommended in the Guide. The dowel size and the reinforcement (JRCP only) were also designed as recommended. The designs were developed at reliability levels of 50, 80, and 90 percent.

These designs were then evaluated using the deterioration models developed from the NCHRP 1-19 data base for pumping, faulting, cracking, joint deterioration, and PSI using the PREDICT program (written in Microsoft BASIC language for the IBM personal computer by M. I. Darter). These models

TABLE 2 SPECIFIC CLIMATIC DATA IN FOUR CLIMATIC ZONES AVERAGED FROM COPES DATABASE

JPCP:	Climatic Zones	Dry - Freeze	Wet - Freeze
	Annual Precipitation, cms	40	84
	Freezing Index, degree-days	250	625
	Mean Temperature, °C	11	11
	* Temperature Range, °C	41	41
	Climatic Zones	Dry - Nonfreeze	Wet - Nonfreeze
	Annual Precipitation, cms	40	120
	Freezing Index, degree-days	0	0
	Mean Temperature, °C	19	19
	Temperature Range, °C	25	30
JRCP:	Climatic Zones	Dry - Freeze	Wet - Freeze
	Annual Precipitation, cms	55	78
	Freezing Index, degree-days	1125	1125
	Mean Temperature, °C	8	8
	Temperature Range, °C	45	43
	Climatic Zones	Dry - Nonfreeze	Wet - Nonfreeze
	Annual Precipitation, cms	-**	120
	Freezing Index, degree-days	-	0
	Mean Temperature, °C	-	17
	Temperature Range, °C	-	34

* Temperature Range = average daily maximum air temperature in July
 minus average daily minimum air temperature in
 January, °C

** No JRCP section is available in dry-nonfreeze climate in COPES.

TABLE 3 DESIGN INPUT PARAMETERS FOR AASHTO PERFORMANCE EQUATION FOR COPES FOUR CLIMATIC ZONES

Parameter	JPCP	JRCP
Reliability level, %	50/80/90	50/80/90
Design period, years	20	20
Traffic, million 18-kip ESAL	15	15
* Subgrade soil type	fine/coarse	fine/coarse
** Subbase type	4" CTB	6" granular
k-value @ top of subbase, pci	300/590	200/420
Initial serviceability	4.5	4.5
Terminal serviceability	2.5	2.5
*** Modulus of rupture, psi	650	650
Concrete E value, psi	4,000,000	4,000,000
Joint spacing, ft	15	27/40
Dowels at joint	yes/no	yes
J factor	3.2/4.1	3.2

* Subgrade $M_R = 3,000$ psi for fine-grained soil and 7,000 psi for coarse-grained soil.

** Subbase E = 1,000,000 psi for CTB and 30,000 psi for granular material.

*** Third-point loading, at 28 days

TABLE 4 CLIMATIC DESIGN INPUTS FOR AASHTO GUIDE FOR COPES FOUR CLIMATIC ZONES

JPCP	Climatic zones	Dry - Freeze				Wet - Freeze			
		fine		coarse		fine		coarse	
	Subgrade soil type	no	yes	no	yes	no	yes	no	yes
	Dowel bars	.95	.95	1.13	1.13	.85	.85	1.05	1.05
	C _d value	.5	.5	.25	.25	1.0	1.0	.5	.5
	LS factor	175	175	400	400	100	100	290	290
*	Corrected k-value								

	Climatic zones	Dry - Nonfreeze				Wet - Nonfreeze			
		fine		coarse		fine		coarse	
	Subgrade soil type	no	yes	no	yes	no	yes	no	yes
	Dowel bars	.95	.95	1.13	1.13	.8	.8	1.0	1.0
	C _d value	.5	.5	.25	.25	1.0	1.0	.5	.5
	LS factor	175	175	400	400	100	100	290	290
	Corrected k-value								

JRCP	Climatic zones	Dry - Freeze		Wet - Freeze	
		fine	coarse	fine	coarse
	Subgrade soil type	yes	yes	yes	yes
	Dowel bars	.95	1.13	.85	1.05
	C _d value	.5	.25	1.0	.5
	LS factor	120	300	70	230
	Corrected k-value				

	Climatic zones	Dry - Nonfreeze		Wet - Nonfreeze	
		fine	coarse	fine	coarse
	Subgrade soil type	yes	yes	yes	yes
	Dowel bars	- **	-	.8	1.0
	C _d value	-	-	1.0	.5
	LS factor	-	-	70	230
	Corrected k-value				

* k-value in pci.

** No JRCP section is available in dry-nonfreeze climate in COPES.

represent the data base mathematically. They provide average projections with about one-half the actual showing worse deterioration and the other half showing less deterioration.

The critical level for each kind of deterioration mentioned above that normally generates the need for rehabilitation is as follows:

Deterioration	JPCP	JRCP
Pumping	1 (low severity)	1 (low severity)
Faulting (in.)	0.13	0.26
Cracking (ft/mile)	800 (all severities)	850 (medium and high severity)
Joint deterioration (joints/mile)	55 (mostly medium severity)	27 (mostly high severity)
PSI	3.0	3.0

These are average values that were determined from the

NCHRP 1-19 data base for sections that were in need of pavement rehabilitation (6).

JPCP

Tables 5 and 6 show the predicted deterioration for each level of reliability for JPCP designs. Figures 5, 6, and 7 illustrate predicted pumping severity, faulting, and slab cracking, respectively.

• Increasing design reliability from 50 to 90 percent increases the thickness of the slab by about 1.5 in. The deterioration prediction models indicate that JPCP slab cracking decreases greatly with this increase in reliability. Pumping and joint faulting decrease a small amount, and terminal PSI increases

TABLE 5 PREDICTIONS FOR JPCP DESIGNS USING AASHTO GUIDE FOR FOUR CLIMATIC ZONES:
50 PERCENT RELIABILITY LEVEL

Climatic zones	Dry-freeze				Wet-freeze			
	fine		coarse		fine		coarse	
Subgrade soil type	0	1.125	0	1.00	0	1.25	0	1.00
Dowel diameter, in	10.6	9.3	9.2	7.9	11.5	10.1	9.8	8.5
Slab thickness, in								
Pumping *	.6	1.3	.6	1.7	2	2.6	1.9	2.8
Faulting, in	.1	.05	.11	.07	.12	.05	.13	.09
Cracking, ft/mile	151	368	249	713	115	315	251	853
Joint deter., jts/mile	12	12	12	12	12	12	12	12
PSI	3.7	3.5	3.6	3.4	3.3	3.0	3.1	2.8

Climatic zones	Dry-nonfreeze				Wet-nonfreeze			
	fine		coarse		fine		coarse	
Subgrade soil type	0	1.125	0	1	0	1.25	0	1.125
Dowel diameter, in	10.6	9.3	9.2	7.9	11.8	10.4	10	8.7
Slab thickness, in								
Pumping	0	.8	0	1.1	.5	1.1	.4	1.3
Faulting, in	.06	0	.07	.03	.06	0	.07	.01
Cracking, ft/mile	51	117	72	144	41	80	70	134
Joint deter., jts/mile	12	12	12	12	12	12	12	12
PSI	4.0	3.8	3.9	3.7	3.6	3.4	3.4	3.2

* Pumping = 1 for low, 2 for medium and 3 for high severity.

Design traffic: 15 million 18-kip ESAL

Design period: 20 years

Subbase type: 4" CTB

Joint spacing: 15 ft

Level of reliability: 50%

a small amount. Joint deterioration does not change, however. Increasing design reliability is an effective way to reduce slab cracking, but not other types of deterioration.

- The use of dowels in transverse joints results in a decrease of about 1.4 in. of slab thickness. The thinner slab develops more cracking, but joint faulting decreases substantially for designs having dowel bars.

- The required slab thickness is reduced about 1.6 in. when the subgrade soil changes from fine-grained to coarse-grained. Even though a coarse-grained soil improves support and sub-drainage, slab thickness is decreased by this extent, which has the overall effect of increasing the amount of slab cracking.

- Pumping is predicted to occur in all climatic regions, but

severe pumping is predicted in the wet-freeze region for these particular designs.

- Transverse joint faulting decreases greatly with the use of dowels. Faulting is greatest in the freeze zones (where without dowels it exceeds the 0.13-in. critical level) and least in the nonfreeze zones. Increased design reliability does not improve faulting very much.

- Slab cracking increases whenever slab thickness is decreased. However, even the maximum predicted cracking is below the critical level for JPCP for all designs.

- Transverse joint deterioration is not affected by design reliability, and joint deterioration for JPCP is minimal.

- Present serviceability ratings at the end of the design life

TABLE 6 PREDICTIONS FOR JPCP DESIGNS USING AASHTO GUIDE FOR FOUR CLIMATIC ZONES:
90 PERCENT RELIABILITY LEVEL

Climatic zones	Dry-freeze				Wet-freeze			
	fine		coarse		fine		coarse	
Subgrade soil type	0	1.375	0	1.125	0	1.375	0	1.25
Dowel diameter, in	12.1	10.7	10.6	9.2	13.1	11.5	11.3	9.8
Slab thickness, in								
Pumping *	0	.6	0	.6	1.5	2.0	1.2	1.9
Faulting, in	.09	0	.1	.05	.11	.02	.12	.05
Cracking, ft/mile	70	143	109	249	49	115	91	251
Joint deter., jts/mile	12	12	12	12	12	12	12	12
PSI	4.0	3.7	3.8	3.6	3.5	3.3	3.3	3.1

Climatic zones	Dry-nonfreeze				Wet-nonfreeze			
	fine		coarse		fine		coarse	
Subgrade soil type	0	1.375	0	1.125	0	1.5	0	1.25
Dowel diameter, in	12.1	10.7	10.6	9.2	13.5	11.9	11.6	10.1
Slab thickness, in								
Pumping	0	0	0	0	0	.4	0	.4
Faulting, in	.05	0	.06	0	.05	0	.06	0
Cracking, ft/mile	25	48	37	72	20	39	34	67
Joint deter., jts/mile	12	12	12	12	12	12	12	12
PSI	4.3	4.0	4.1	3.9	3.9	3.6	3.7	3.4

* Pumping = 1 for low, 2 for medium and 3 for high severity.

Design traffic: 15 million 18-kip ESAL

Design period: 20 years

Subbase type: 4" CTB

Joint spacing: 15 ft

Level of reliability: 90%

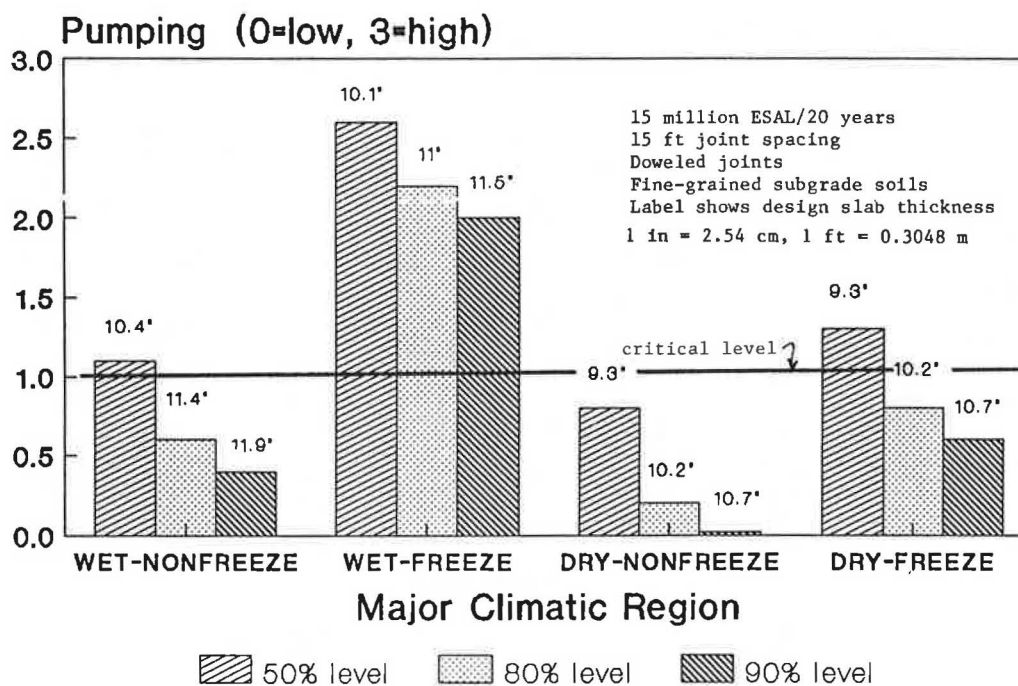


FIGURE 5 Pumping severity by climatic regions at various reliability levels for AASHTO JPCP designs.

are all predicted to be above 3.0 (with one slight exception). Increased design reliability increases terminal serviceability by a small amount.

Overall, deterioration of JPCP generally increases as the climate becomes wetter or colder, even though the design procedure attempts to adjust the design for climate. Deterioration differs significantly among climatic regions. The drainage coefficient, C_d , value had a significant influence on the thickness design.

In general, the results show that the AASHTO Guide provides adequate structural designs for JPCP at the higher design reliability level for this example. JPCP in wet- or dry-freeze regions shows more deterioration (particularly pumping) than other regions. Special design improvements may be necessary in freeze areas.

JRCP

Tables 7 and 8 show the predicted deterioration for each level of reliability for JRCP designs. Figures 8, 9, and 10 illustrate predicted pumping severity, cracking, and joint deterioration, respectively.

- Increasing design reliability from 50 to 90 percent increases the thickness of the slab by about 1.4 in. The deterioration prediction models indicate that JRCP slab cracking, faulting, and pumping decrease substantially with this increase in reliability. Terminal PSI increases somewhat. Joint deterioration does not change, however. Increasing design reliability (and hence slab thickness) is an effective way to reduce slab cracking, but probably not the other types of deterioration.

- All the JRCP sections were designed with dowels. The

presence of dowels replaces approximately 1.4 in. of slab thickness. JRCP with dowels did not show excessive joint faulting.

- At every reliability level, the AASHTO Guide requires thinner slabs for the JRCP on coarse-grained subgrade soil than on fine-grained subgrade soil. For example, at the 50 percent reliability level (Table 8), the required slab thickness for shorter and longer joint spacings varies from 1.3 to 1.7 in. between fine- and coarse-grained subgrade soil designs. Even though a coarse-grained soil improves support and sub-drainage, slab thickness is decreased by this extent, which has the overall effect of increasing the amount of slab cracking.

- Pumping is predicted to occur in all climatic regions, but severe pumping is predicted in both freeze regions.

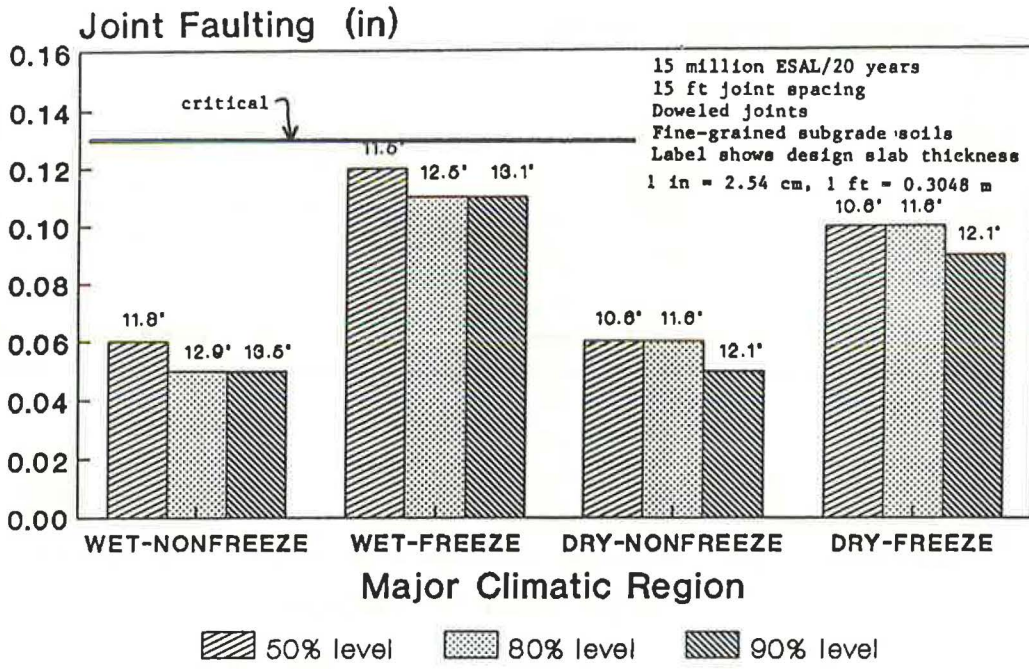
- Faulting is greatest in the freeze zones; however, predicted faulting for all designs was less than critical for JRCP (0.26 in.). Increased design reliability does not improve faulting very much.

- Transverse joint deterioration is not affected by design reliability. It is greatest in the freezing climatic zones where the 40-ft designs are predicted to exceed the critical level of 27 joints per mile. The shorter 27-ft designs do not exceed this critical level of deterioration.

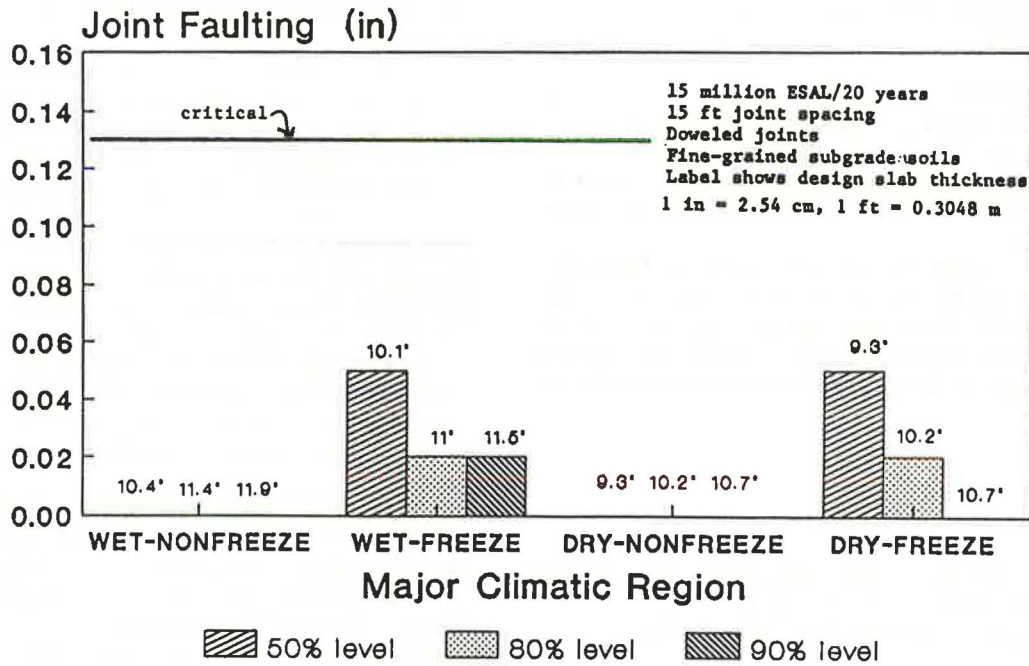
- Transverse cracks are predicted to become a severe problem for almost all of the design cases. The increased slab thickness for increased reliability helps, but does not solve the problem.

- The PSI at the end of the design life falls below the critical level of 3.0 in several cases at the 50 percent reliability level, but not at the 80 or 90 percent levels. Increased reliability increases terminal serviceability somewhat.

Overall, deterioration of JRCP increases as the climate becomes wetter or colder, even though the procedure attempts



(a) undoweled



(b) doweled

FIGURE 6 JPCP faulting predictions by climatic regions for undoweled and doweled pavements at various reliability levels.

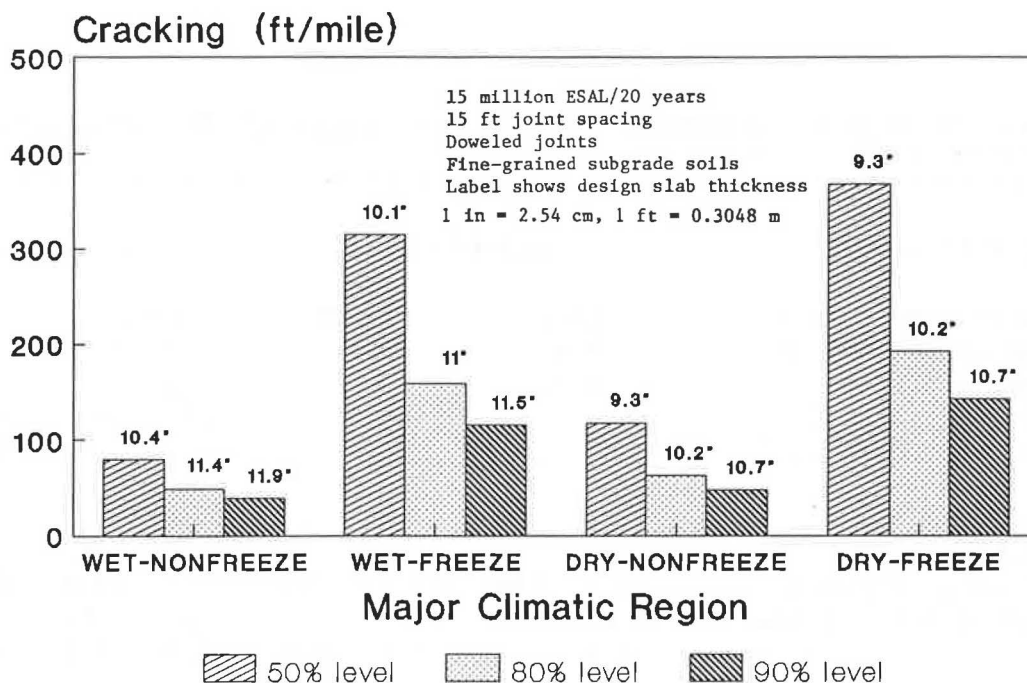


FIGURE 7 Predicted slab cracking by climatic regions at various reliability levels for AASHTO JPCP designs.

to adjust the design for climate. Significant differences in performance exist between climatic regions. The drainage coefficient, C_d , value had a significant influence on the thickness design.

In general, the prediction results show that the new Guide provides only fair structural designs for JRCF for the four climatic regions and worse designs for the wet-freeze region. This is evidenced by excessive crack deterioration, pumping, and joint deterioration. The AASHTO Guide does not provide adequate structural designs for JRCF on coarse-grained subgrade soils. However, a comparison of the results from the predicted vs. actual ESAL evaluation, which simulates the original AASHTO Road Test performance model, with these results shows that the adjustment factors provided in the new AASHTO Guide are somewhat effective in improving the JRCF designs.

Some components of the pavement, however, showed serious failure and did not improve at the higher levels of reliability. This included joint deterioration with 40-ft or more joint spacing. For example, the JRCF with 27-ft joint spacing has less faulting than with 40-ft joint spacing. The 40-ft joint spacing also results in serious joint deterioration whereas the 27-ft joint spacing gives much better performance. As many as 61 deteriorated joints per mile were predicted for the 40-ft joint spacing in the freeze climates (Figures 8, 9, and 10). The AASHTO Guide does not provide adequate, coherent guidance on joint design.

CONCLUSIONS

The conceptual evaluation of the 1986 AASHTO Design Guide indicated that, since the AASHTO structural design model

was empirically derived in a single climate, with a single set of materials, and a short traffic loading period, it has many potential inherent weaknesses and limitations. Major ones include subdrainage consideration, climatic extrapolation, long-term climatic effects, joint design, and reinforcement design.

The analytical evaluation generally supports the conceptual evaluation. There is serious prediction error associated with the original AASHTO equations when they are used for differing climates and for different pavement designs. The design adjustment factors included in the new AASHTO Guide are beneficial, particularly the design reliability. However, the joint design, loss-of-support, and drainage coefficient recommendations are still particularly deficient. JPCP designs appear to be much more resistant to the long-term damaging effects of time and climate than JRCF designs.

The overall conclusion is that there are several deficiencies in the new Guide. It is recommended that each agency that adopts the guide conduct an evaluation similar to the example reported herein to determine the general deterioration of pavements existing on their highways. Recommendations for joint design must be developed by each agency since none exist in the Guide. Levels of reliability higher than 50 percent must be considered to provide adequate designs. The 90 percent level studied here showed reasonable results for JPCP. Improved reinforcement design is needed for JRCF to control crack deterioration.

ACKNOWLEDGMENT

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TABLE 7 PREDICTIONS FOR JRCP DESIGNS USING AASHTO GUIDE FOR FOUR CLIMATIC ZONES:
50 PERCENT RELIABILITY LEVEL

Climatic zones	Dry-freeze				Wet-freeze			
	fine		coarse		fine		coarse	
Subgrade soil type	9.4		8.1		10.2		8.6	
Slab thickness, in	1.125		1		1.25		1.125	
Dowel diameter, in	27	40	27	40	27	40	27	40
Joint spacing, ft	.047	.069	.04	.06	.051	.075	.043	.064
Area of steel, sq in/ft	1.6	1.6	3	3	2.2	2.2	3	3
Pumping *	.07	.12	.16	.17	.06	.11	.09	.14
Faulting, in	1322	1300	2932	2904	1166	1169	2437	2432
Cracking, ft/mile	0	61	0	61	0	61	0	61
Joint deter., jts/mile	3.1	3.0	2.7	2.6	3.3	2.9	3.2	2.8
PSI								

Climatic zones	Dry-nonfreeze				Wet-nonfreeze			
	- **				fine		coarse	
Subgrade soil type	-				10.6		8.9	
Slab thickness, in	-				1.375		1.125	
Dowel diameter, in	-				27	40	27	40
Joint spacing, ft	-				.053	.078	.044	.066
Area of steel, sq in/ft	-				1.1	1.1	2.1	2.1
Pumping	-				0	.05	.03	.08
Faulting, in	-				942	944	1636	1639
Cracking, ft/mile	-				0	35	0	35
Joint deter., jts/mile	-				3.3	3.2	2.9	2.9
PSI	-							

* Pumping = 1 for low, 2 for medium and 3 for high severity.

** No JRCP section is available in dry-nonfreeze climate in COPES.

Design traffic: 15 million 18-kip ESAL

Design period: 20 years

Subbase type: 6" granular

Level of reliability: 50%

TABLE 8 PREDICTIONS FOR JRCP DESIGNS USING AASHTO GUIDE FOR FOUR CLIMATIC ZONES:
90 PERCENT RELIABILITY LEVEL

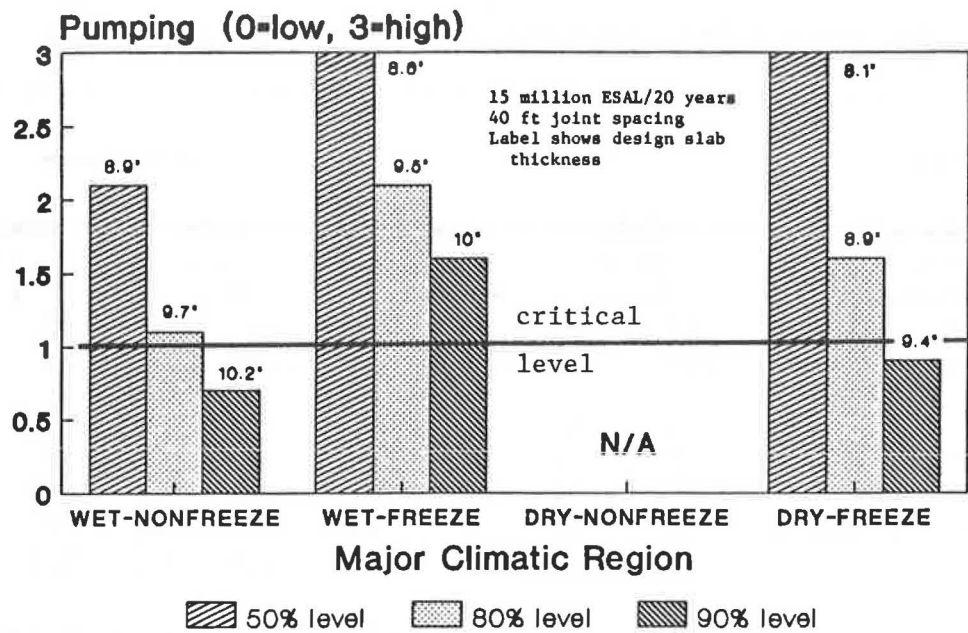
Climatic zones	Dry-freeze				Wet-freeze			
	fine		coarse		fine		coarse	
Subgrade soil type	fine		coarse		fine		coarse	
Slab thickness, in	10.8		9.4		11.7		10	
Dowel diameter, in	1.375		1.125		1.5		1.25	
Joint spacing, ft	27	40	27	40	27	40	27	40
Area of steel, sq in/ft	.054	.08	.047	.069	.058	.086	.05	.074
Pumping *	.5	.5	.9	.9	1.5	1.5	1.6	1.6
Faulting, in	0	.05	0	.05	0	.05	0	.03
Cracking, ft/mile	906	906	1265	1271	822	822	1099	1100
Joint deter., jts/mile	0	61	0	61	0	61	0	61
PSI	3.6	3.5	3.4	3.3	3.5	3.4	3.3	3.2

Climatic zones	Dry-nonfreeze				Wet-nonfreeze			
	fine		coarse		fine		coarse	
Subgrade soil type	- **				fine		coarse	
Slab thickness, in	-				12		10.2	
Dowel diameter, in	-				1.5		1.25	
Joint spacing, ft	-				27	40	27	40
Area of steel, sq in/ft	-				.06	.089	.051	.075
Pumping	-				.6	.6	.7	.7
Faulting, in	-				0	.03	0	0
Cracking, ft/mile	-				775	775	1017	1020
Joint deter., jts/mile	-				0	35	0	35
PSI	-				3.6	3.5	3.4	3.3

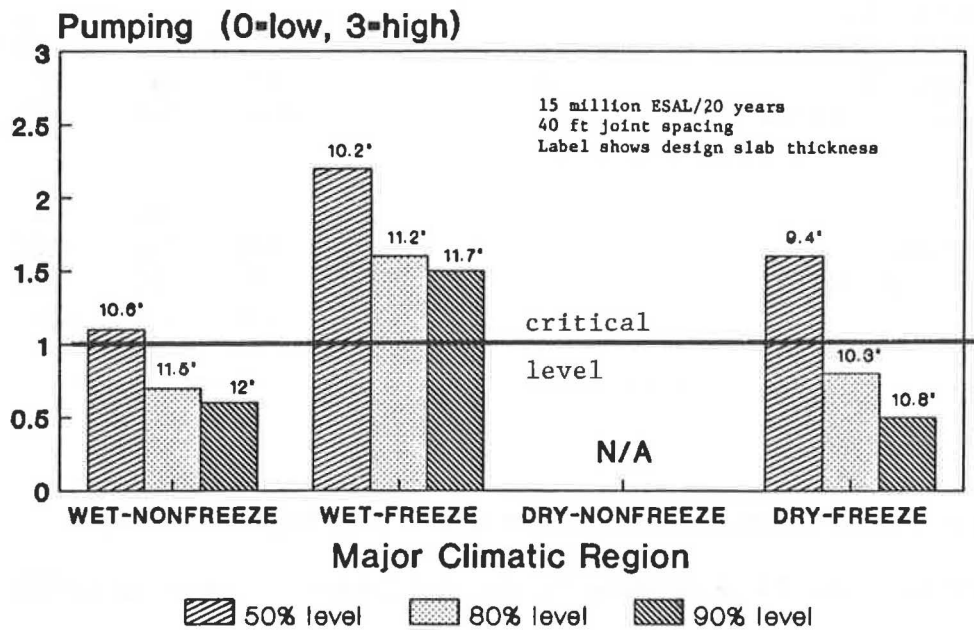
* Pumping = 1 for low, 2 for medium and 3 for high severity.

** No JRCP section is available in dry-nonfreeze climate in COPES.

Design traffic: 15 million 18-kip ESAL
 Design period: 20 years
 Subbase type: 6" granular
 Level of reliability: 90%

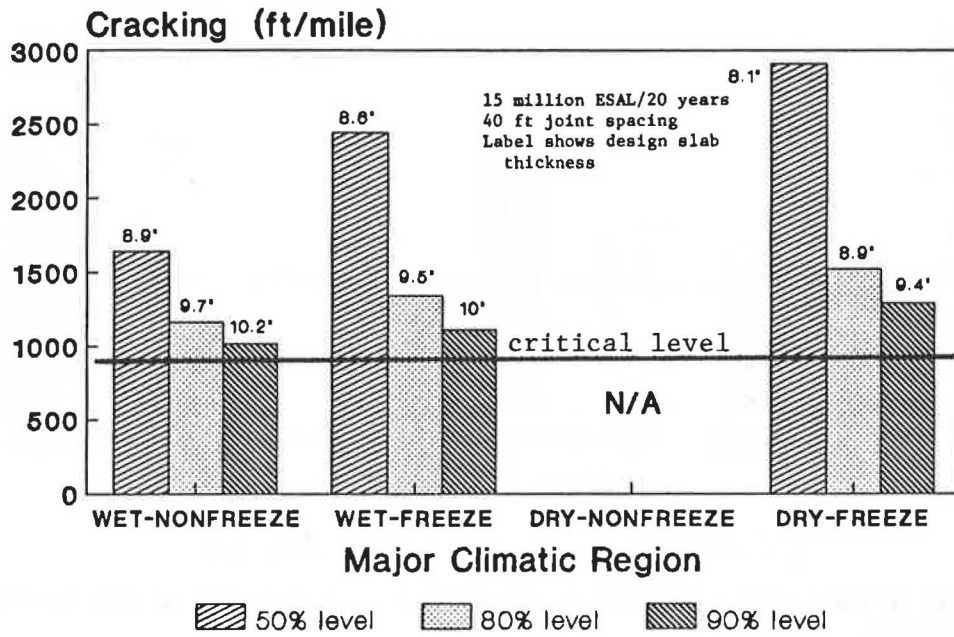


(a) coarse-grained subgrade soil

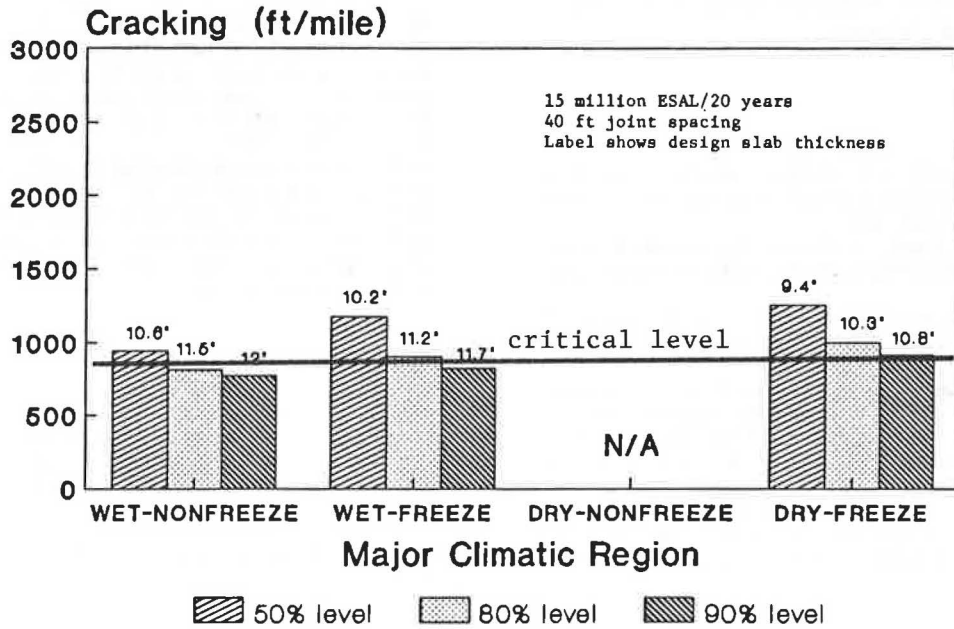


(b) fine-grained subgrade soil

FIGURE 8 Pumping severity by climatic regions at various reliability levels for AASHTO JRCP designs.



(a) coarse-grained subgrade soil



(b) fine-grained subgrade soil

FIGURE 9 Predicted slab cracking by climatic regions at various reliability levels for AASHTO JRC designs.

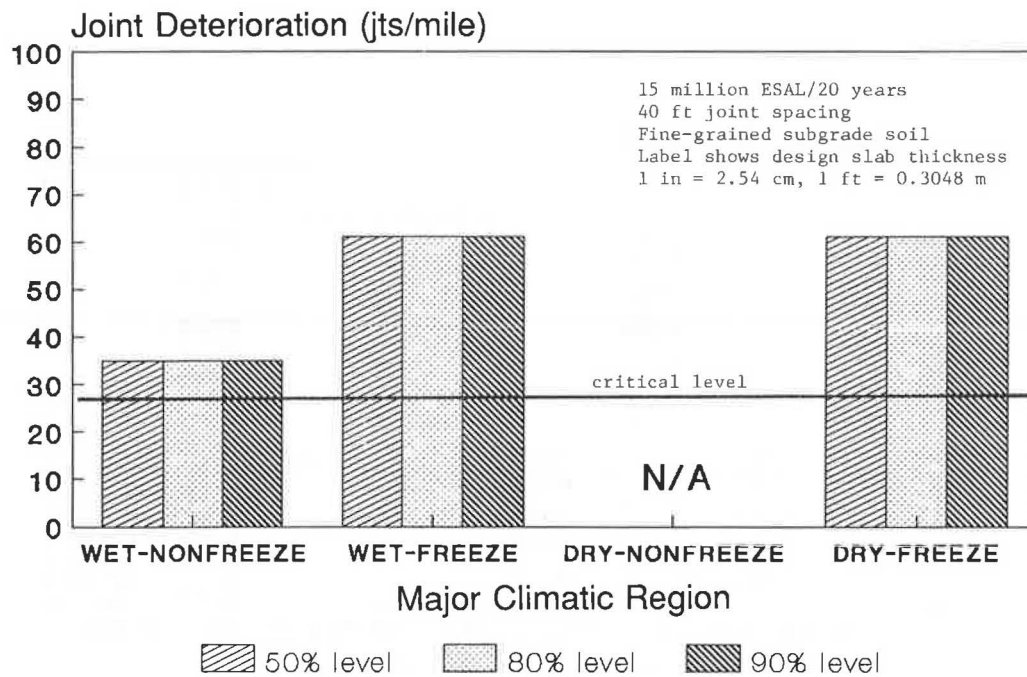


FIGURE 10 Predicted joint deterioration by climatic regions at various reliability levels for AASHTO JRCP designs.

at Urbana-Champaign. The authors express their appreciation to Stephen Forster of the Federal Highway Administration for his assistance and comments.

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