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FOREWORD

The recent concern among pavement engineers about the reliability and performance of continuously reinforced concrete pavement makes the reports in this RECORD very timely. Three papers cover the field performance of CRCP, and 3 others provide concepts for more realistic design analysis of concrete pavements. The latter 3 papers apply to all types of rigid pavements. The reports will be of interest to practicing engineers and researchers.

A field study of CRCP in Indiana by Faiz and Yoder provides excellent information to permit a designer to make qualitative decisions. The results showed that subbase type, methods of steel placement, steel fabrication, concrete slump, and traffic have a significant effect on CRCP performance. Performance is defined in terms of failures, close transverse crack spacing, spalled cracks, and edge pumping.

McGhee describes the performance of CRC pavements in Virginia and the transverse cracking characteristics. He concludes that the performance is good except for one project. An interesting observation from a core study was that the cracks are discontinuous in area of steel and grow progressively wider toward the top and bottom of the slab. He concludes that a steel corrosion problem does not exist in pavements having normal crack patterns.

Teng and Coley report on a 5-year study of CRCP performance in Mississippi. The effect of numerous variables such as paving temperature, traffic loads, degree of vibration, and stabilized base on performance is covered. The studies cover over 900 two-lane miles of CRCP.

Huang and Wang used finite-element theory to analyze the effect of partial slab support on the stresses in the pavement. The validity of the approach is validated by using AASHO Road Test data. The paper should be of interest to those experiencing edge pumping such as described in the first paper.

Buick and Oppenlander report on a sensitivity study of rigid pavement design methods of AASHO, PCA, and Corp of Engineers. They observed that traffic load and flexural strength of the concrete are the most influential factors in the design of rigid pavement.

Darter et al. present a methodology for selecting an optional pavement design; reliability, performance, and costs are considered. The authors emphasize that there are numerous pavement design strategies that will give satisfactory performance during an analysis period. Thus, the administration must be provided with rational information in order to make a selection.

—B. F. McCullough

FACTORS INFLUENCING THE PERFORMANCE OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

Asif Faiz and Eldon J. Yoder, Purdue University

A statewide condition survey of continuously reinforced concrete (CRC) pavements was conducted in Indiana in 1972 to evaluate the effects of subbase and subgrade type, the methods of paving, steel placement and steel fabrication, concrete slump, and traffic on CRC pavement performance. The measures of performance were extent of failures, parallel cracks with less than 30-in. (76-cm) crack spacing, random cracks, spalled cracks, and edge pumping. The results show that subbase type, methods of steel placement and steel fabrication, concrete slump, and traffic significantly influence CRC pavement performance. Gravel subbases showed poorer performance than crushed stone and bituminous stabilized subbases. Better performance was indicated where deformed wire fabric or loose bars were used than where tied bar mats were used. Depressed steel performed better than steel preset on chairs. The data showed little difference between performance of pavements that were slip-formed and those that were side-formed. Relative to good performance, an optimum range of concrete slump between 2.0 and 2.5 in. (5.0 to 6.5 cm) was indicated. Distress of CRC pavements is associated with traffic. Most of the pumping was observed on pavements with gravel subbases, though some pumping was also indicated where bituminous-stabilized or crushed-stone subbases were used.

•DURING the past several years use of continuously reinforced concrete pavement (CRCP) has increased considerably. The first experimental continuously reinforced concrete pavement was built in 1938 on US-40 near Stilesville, Indiana. During the next 20 years, a number of research-oriented CRCP projects were built at various locations in the United States. Experimental test sections were constructed in Illinois and New Jersey in 1947 and in California in 1949. In 1951, portions of the Fort Worth freeways in Texas were constructed with continuously reinforced concrete; other CRC pavements were built in Texas in 1955 and 1957. In addition, 2 continuously reinforced concrete projects were constructed in Pennsylvania in 1956 and 1957.

As of 1958, there were 79 miles (127 km) of equivalent 2-lane CRC pavement in the United States. Since then the use of CRC pavements in highway construction has increased; more than 10,000 miles (16 000 km) of equivalent 2-lane pavement were in use or under contract in 33 states at the end of 1971 (10).

Outside the United States, a number of countries have built CRC pavements. Notable among these are Belgium, West Germany, the Netherlands, Sweden, and Switzerland. The Great Britain Road Research Laboratory investigated the use of CRC bases under asphalt surface courses (3). Belgium built its first experimental CRC pavement in 1950 and recently decided to undertake such construction over 81 miles (130 km) of freeway (8).

One of the primary reasons for constructing this type of pavement is that CRC pavements have a better riding quality than jointed concrete pavements, and in most cases

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these pavements offer an effective means of serving heavy traffic with a minimum of interruption for routine maintenance and repairs.

Figure 1 shows the extent of CRC pavement constructed in Indiana up to the summer of 1972. The first pavement was built on an experimental basis in 1938. Several short sections of pavement were constructed in the mid-1960s. During the past several years many additional miles of CRC pavement have been built, primarily on the Interstate System. The increase in the use of CRC pavements in Indiana is shown in Figure 2.

Most of the pavements constructed in Indiana are 9 in. (23 cm) thick, although some have been constructed 7 and 8 in. (18 and 20 cm) in thickness. For the most part, nonstabilized granular subbases have been used under the pavement, although in recent years the trend has been toward the use of asphalt-treated subbases in most situations.

Various types of steel placement and construction (formed or slip-formed) have been used. The percentage of steel used has been 0.6 percent of the cross-sectional area, irrespective of the other factors of design.

STATEWIDE CONDITION SURVEY OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS IN INDIANA

To evaluate the performance of CRC pavements in Indiana, a statewide condition survey was conducted in late 1972. The field survey was a cooperative venture in which a study group from Purdue University was assisted by personnel from the Research and Training Center and the Crawfordsville District Office of the Indiana State Highway Commission. A sampling procedure was used to design the field survey, and statistical methods were used to analyze the resulting data.

STUDY DESIGN

The intent of the study design was to ensure the inclusion in the study of every CRCP contract that had been completed up to the time of the survey. A further purpose was to provide an inference space for the proposed analysis that would encompass all the factors under investigation.

Sampling Procedure

A stratified random sample of CRC pavements was used in the field survey. Stratified random sampling is a plan by which the population under consideration (in this case, all the CRCP contracts in Indiana) is divided into strata or classes according to some principle significant to the projected analysis. This is followed by sampling within each class as if it were a separate universe. The aim in stratification is to break up the population into classes that are fundamentally different in respect to the average or level of some quality characteristics (6, 7).

Such a sampling scheme is superior to a simple random sample in that the inclusion of all independent factors to be evaluated in the study is guaranteed. This vastly improves the inference space of the desired analysis.

Only one simple random sample was obtained from each stratum or class. Such a sample or unit of evaluation was designated as a field survey section. Each field survey section was a 5,000-ft (1524-m) length of pavement. The location, relative to the direction of lanes, and beginning of each section were selected from the total length of CRC pavement in each stratum by the use of random number tables. Care was taken that a randomly selected pavement length was located approximately 200 to 300 ft (60 to 90 m) away from the exact end or beginning of a construction contract.

The survey sections were stratified on the basis of the following factors: contract, method of paving, method of steel placement, method of steel fabrication, type of subbase, and type of subgrade. These factors are described in detail in the section on statistical design. Data relative to these factors were obtained from construction survey records. In addition, information pertaining to concrete slump, date of paving, and date a section was opened to traffic was also taken from construction records.

Most of the pavements were 9 in. (23 cm) thick, although several were 8 in. (20 cm) thick and 9 were 7 in. (18 cm) thick.

Figure 1. CRC pavements in Indiana in summer of 1972.

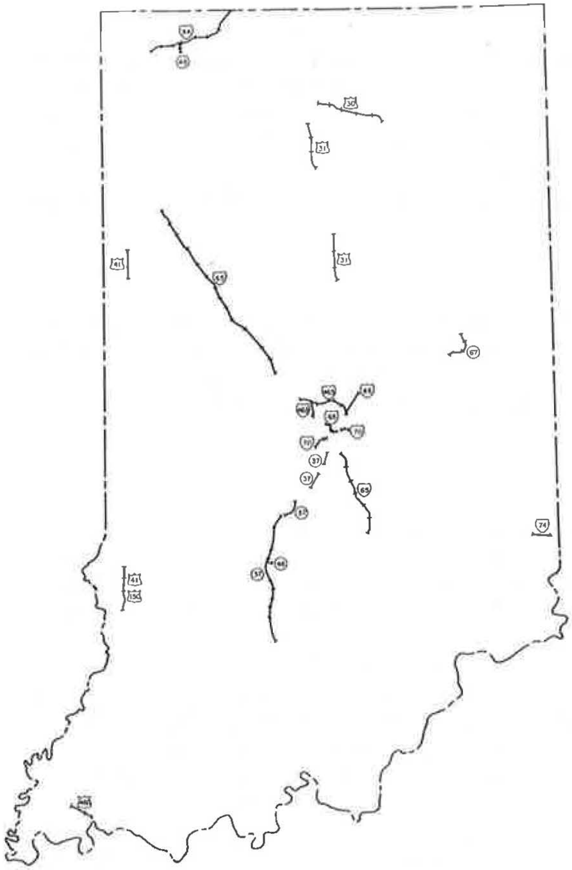
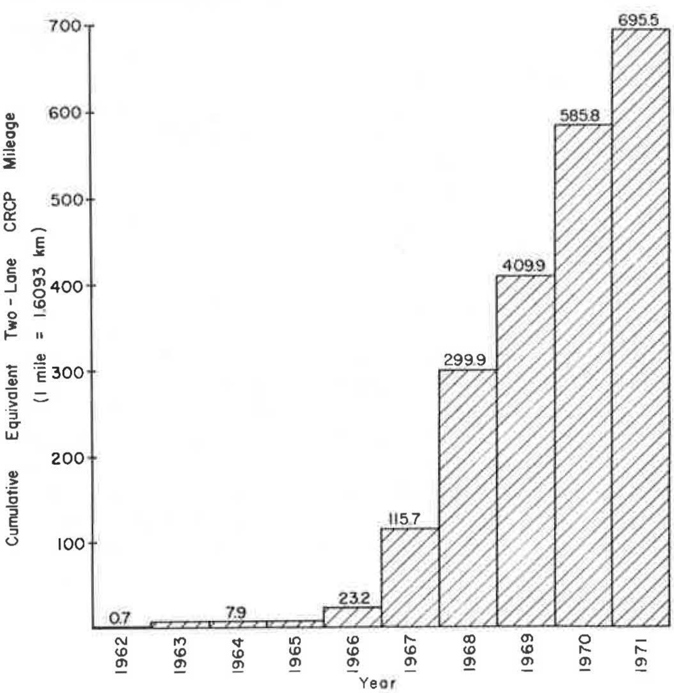


Figure 2. Use of CRCP in Indiana from 1962 to 1971.



In certain cases, more than one survey section was sampled within a particular contract. This apparent duplication resulted whenever a contract crossed more than one level of any other stratification factor. For example, if 2 subgrade types occurred over one contract, 2 sections were included in the survey. Similarly, 2 sections were surveyed if 2 different methods of steel placement were used within a particular contract. Consequently, 89 CRCP sections were used in the survey.

A provision was made in the study design so that some of the sections would be surveyed twice by different survey teams.

Statistical Design

To study the factors influencing the performance of CRC pavements, we used a $2 \times 2 \times 3 \times 4 \times 2$ factorial design with unequal subclass frequencies. A number of covariates or concomitant variables were superimposed on the factorial. The layout of the statistical design is shown in Figure 3, which also indicates the independent factors and their corresponding levels selected for this investigation.

Independent Factors

Method of Paving—This factor had 2 levels: side-formed and slip-formed.

Method of Steel Placement—The method of placing steel reinforcement was subdivided into 2 categories: preset on chairs and placed by mechanical means. The latter was usually accomplished by placing the reinforcement on top of plastic concrete and depressing it to the prescribed depth by a machine that imparts pressure and vibration. Hence, the 2 levels of this factor were labeled as "chairs" and "depressor." Concise description of methods of steel placement are given in other reports (1, 5, 13).

Method of Steel Fabrication—The 3 kinds of steel reinforcement used in CRC pavements formed the 3 levels of this factor: loose reinforcing bars, tied bar mats, and welded deformed wire fabric. The amount of longitudinal steel used was 0.6 percent of the pavement cross-sectional area irrespective of other design factors.

In case of loose bars and tied bar mats, longitudinal reinforcement consisted of No. 5 bars with a center-to-center (c. to c.) spacing of 5.5 in. (14 cm) for a 9-in. (23-cm) thick pavement and a c. to c. spacing of 6.25 in. (16 cm) for 7- and 8-in. (18- and 20-cm) thick pavements. Use of No. 4 bars with a c. to c. spacing of 4 in. (10 cm) for an 8-in. (20-cm) thick pavement and a c. to c. spacing of 4.5 in. (11.4 cm) for a 7-in. (18-cm) thick pavement was also permitted. For transverse reinforcement, No. 4 bars with c. to c. spacing of 3 ft (0.9 m) were used irrespective of pavement thickness. In some cases where steel reinforcement was mechanically placed, transverse steel was omitted. According to Indiana specifications (11), welding of intersections is not permitted on tied bar mats. Furthermore, the mats may be assembled either inside or outside the forms. The reinforcement was required to be deformed billet steel bars.

The longitudinal reinforcement in welded deformed wire fabric consisted of wires of sizes D-16.8, D-19.2, and D-21.6 at 4-in. (10-cm) c. to c. spacing for 7-in. (18-cm), 8-in. (20-cm), and 9-in. (23-cm) thick pavements respectively. For transverse reinforcement, wires of sizes D-4 to D-6 with a c. to c. spacing varying from 12 to 16 in. (30.5 to 41 cm) were used.

Type of Subbase—A variety of subbase materials have been used under CRC pavements in Indiana: gravel, air-cooled or granulated blast furnace slag, crushed stone, and plant-mixed bituminous stabilized aggregate (stone, gravel, or slag) with an asphalt content of 2.5 to 4.5 percent. Both asphalt cement and asphalt emulsions have been used as stabilizing agents. These materials constituted the 4 levels of this factor.

Type of Subgrade—Subgrades were classified into 2 types: fine-grained and granular. This information was obtained from aerial photographic strip maps and an engineering soils map of Indiana (12). The CRC pavements in Indiana traverse a variety of landforms. Of these physiographic units, ground moraines, ridge moraines, lacustrine lake-bed deposits, residual deposits, floodplains, and alluvial deposits were classified as fine-grained parent materials. Gravel terraces, eskers, glacial outwash deposits, beach ridges, and sand dunes were considered as granular parent materials.

Covariates or Concomitant Variables

Covariates or concomitant variables are used in statistical designs to increase the precision of the statistical experiment by removing potential sources of bias in the experiment. In this investigation, it was considered necessary to incorporate some property of concrete and some measure of traffic load applications, for these variables have a considerable effect on distress in concrete pavements. The 2 covariates used in the statistical design were (a) concrete slump measured in inches and obtained from construction survey records and (b) number of months since a pavement section was opened to traffic. The latter variable was used as an indirect measure of load applications.

Response or Evaluated Variables

The following measures of performance were logged by the field survey teams.

1. The term "defect" was used to define all pavement surface features indicative of a failure. The term included breakups, punch-outs, asphalt patches, and concrete patches.
2. Breakups and punch-outs were counted and also estimated in terms of area.
3. Asphalt and concrete patches were counted and also estimated in terms of area.
4. The number of spalled cracks per survey section were counted in terms of 3 qualitative categories: slightly spalled, moderately spalled, and excessively spalled.
5. Parallel cracks, with spacing less than 30 in. (76 cm), and random cracks were evaluated in terms of linear feet of longitudinal length of pavement.
6. Pumping was estimated in terms of linear feet of pavement section length that showed pumping. Pumping was identified by observing discoloration (mud marks) and wet areas on the shoulder.

Supplementary data included the following:

1. Each breakup or a patch together with its accompanying crack pattern and spalling characteristics was sketched on the survey form, and some of these defects were photographed;
2. Any dates marked on the pavement were recorded;
3. Joints (construction or expansion) were sketched and indicated by a station identification; and
4. Identification features such as bridges and interchanges were indicated by a station identification, and remarks relative to unusual soil characteristics (subgrade) were also recorded.

The primary distress variables included parallel cracks, random cracks, spalled cracks, edge pumping, and defects as noted by patching and the like.

The following response variables were used in the study:

1. Number of defects per survey section, i.e., 5,000 ft (1524 m) of pavement;
2. Number of spalled cracks per survey section;
3. Linear feet of longitudinal pavement section showing random cracks and parallel cracks, having a spacing closer than 30 in. (76 cm); and
4. Linear feet of longitudinal pavement section where edge pumping was indicated.

ANALYSIS AND RESULTS

The data obtained from the statewide CRCP condition survey were statistically analyzed by using a weighted least squares analysis of variance procedure. This procedure was necessitated because of unequal subclass cell frequencies in the data. In this situation, the different comparisons with which the sums of squares are associated become nonorthogonal and usual analysis of variance leads to biased test procedures.

The ANOVA results reported in this study were obtained by using the Least Squares and Maximum Likelihood General Purpose Program, a computer program at the Purdue University Computer Center. This program uses a general weighted least squares procedure (9) and can be used for missing value problems where cell frequencies are unequal and also where data are not available for certain subclasses. The program only handles main effects and 2-factor interactions, but has provisions for incorporating covariates (concomitant variables) in the analysis.

The following analysis of variance model was used:

$$Y_{ijklmp} = \mu + A_i + B_j + C_k + D_l + F_m + AB_{ij} + AC_{ik} + AD_{il}$$
$$+ AF_{im} + BC_{jk} + BD_{jl} + BF_{jm} + CD_{kl} + CF_{km} + DF_{lm}$$
$$+ \beta_1(S_{ijklmp} - \overline{S}) + \beta_2(T_{ijklmp} - \overline{T}) + \epsilon_{(ijklm)p}$$

where

Y_{ijklmp} = dependent variable, e.g., number of defects;
μ = true mean effect for the population;
A_i = true effect of method of paving (slip-formed versus side-formed);
B_j = true effect of method of steel placement (depressor versus chairs);
C_k = true effect of method of steel fabrication (bar mats versus wire fabric versus loose bars);
D_l = true effect of type of subbase (bituminous-stabilized versus crushed-stone versus slag versus gravel);
F_m = true effect of subgrade soil (granular versus fine-grained);
S_{ijklmp} = linear effect of covariate, slump (in.);
T_{ijklmp} = linear effect of covariate, number of months of traffic;
β_1, β_2 = regression coefficients;
$\overline{S}, \overline{T}$ = mean values of slump and traffic respectively; and
$\epsilon_{(ijklm)p}$ = true error, NID (o, σ^2).

The other terms denote the 2-factor interactions among the factors A, B, C, D, and F. The subscripts assume the following values:

$i = 1, 2;$
$j = 1, 2;$
$k = 1, 2, 3;$
$l = 1, 2, 3, 4;$
$m = 1, 2;$ and
$p = 0$ (missing value) or $1, 2, \ldots, n_{ijklm}$ (unequal subclass numbers).

The model does not take into consideration 3-factor and higher order interactions owing to computer program limitations. Consequently, these interaction effects are confounded with the error effect in this formulation.

A square-root transformation was applied to the data to satisfy the requirement of homogeneity of variance, a basic assumption underlying the analysis of variance procedure. The results of the Foster-Burr Q-test (4) used for testing homogeneity of variance are given in Table 1.

In the analysis of variance, interaction effects and corresponding main effects that were nonsignificant at an α-level of 0.25 were pooled with the error effect, and tests of significance were made by using the pooled error term (2).

Tables 2, 3, 4, and 5 give the results of the analysis of variance. The dependent variables used in the analysis were as follows:

1. Square root of number of defects (asphalt patches, concrete patches, and breakups) per section (Table 2);
2. Square root of number of asphalt patches and breakups per section (Table 3);
3. Square root of number of spalled cracks per section, excluding slightly spalled and excessively spalled cracks (Table 4); and
4. Length of pavement section showing random cracks plus parallel cracks with less than 30 in. (76 cm) spacing, in feet per section (Table 5).

The section length was 5,000 ft (1524 m), and the number of observations was 95.

Figure 3. Factorial design for study of factors influencing CRCP performance.

Type of Subgrade	Type of Subbase	Slipformed, Chairs, Loose Bars	Slipformed, Chairs, Bar Mats	Slipformed, Chairs, Wire Fabric	Slipformed, Depressor, Loose Bars	Slipformed, Depressor, Bar Mats	Slipformed, Depressor, Wire Fabric	Side Formed, Chairs, Loose Bars	Side Formed, Chairs, Bar Mats	Side Formed, Chairs, Wire Fabric	Side Formed, Depressor, Loose Bars	Side Formed, Depressor, Bar Mats	Side Formed, Depressor, Wire Fabric
Fine Grained	Bituminous Stabilized	9,8	8		9,8	8,8	8,8,8						
Fine Grained	Gravel	9,9,9, 8	9,9,9, 9,9,9, 9,8,8, 7,7	9,9,9, 8,8,8		9	9,9,9, 8			9,8,8, 8		8	9,9,9, 9,9,9, 8
Fine Grained	Crushed Stone	9,7,7, 7,7,7, 7		9,9,8			9,8,8, 7		9				9
Fine Grained	Slag					9,8,8, 8	9						9
Granular	Bituminous Stabilized	8			8,8	8,8							
Granular	Gravel		9	8		8	9			9		8	9,9
Granular	Crushed Stone			9			9		9				9
Granular	Slag					8,8							

Method of Paving; Method of Steel Placement; Method of Steel Fabrication; Type of Subbase; Type of Subgrade

Numbers in cells denote thickness of CRC pavement in inches.

Table 1. Foster-Burr test for homogeneity of variance.

Response Variable	Avg DF per Sample	Degrees of Freedom	Number of Samples	$Q_{(calculated)}$	$Q_{3,20,0.001}$	Homogeneity of Variance at $\alpha = 0.001$
Defects per section						
Number	3	3	20	0.4205	0.146	Reject
Square root	3	3	20	0.1011	0.146	Accept
Asphalt patches and breakups per section						
Number	3	3	20	0.2273	0.146	Reject
Square root	3	3	20	0.1308	0.146	Accept
Spalled cracks per section						
Number	3	3	20	0.1609	0.146	Reject
Square root	3	3	20	0.0925	0.146	Accept
L_{RC+PC} [a]	3	3	20	0.0913	0.146	Accept

[a]Length of pavement section showing random cracks plus parallel cracks with less than 30-in. (76-cm) spacing, in feet per section.

Table 2. Least squares analysis of variance of number of defects per section.

Source	DF	Sums of Squares	Mean Squares	F	$F_{0.05}$	Significant at $\alpha = 0.05$
Total (uncorrected)	95	102.756				
Main effects						
A_i	1	0.971	0.971	1.77	3.98	–
B_j	1	0.112	0.112	0.21	3.98	–
C_k	2	3.787	1.894	3.46	3.12	Yes
D_l	3	6.527	2.176	3.97	2.74	Yes
F_m	1	0.440	0.440	0.80	3.98	–
Interaction effects						
AB_{ij}	1	1.103	1.103	2.01	3.98	–
AC_{ik}	1	1.073	1.073	1.96	3.98	–
CD_{kl}	5	3.717	0.743	1.36	2.35	–
CF_{km}	2	2.529	1.265	2.31	3.12	–
Covariate effects						
S_{ijklmp}	1	3.114	3.114	5.68	3.98	Yes
T_{ijklmp}	1	1.036	1.036	1.89	3.98	–
$\epsilon_{(ijklm)p}$	75	41.117	0.548			

The extent of pavement distress was evaluated primarily in terms of number of defects. Asphalt patches and breakups were considered separately, for they manifest recent pavement distress. Concrete patches were included in only one evaluation scheme because it was not known exactly when the concrete patches were placed.

Factors Affecting Pavement Distress as Evaluated by Number of Defects per Section

The results of the analysis of variance given in Tables 2 and 3 indicate that

1. The method of steel fabrication and subbase type together with concrete slump have a significant effect on pavement distress as evaluated by the number of defects (concrete patches, asphalt patches, and breakups) observed per section;
2. The method of steel fabrication, the type of subbase, traffic, and the interaction between the methods of paving and placing steel reinforcement have a signfficant influence on pavement distress as determined by the number of asphalt patches and breakups observed per section; and
3. Irrespective of the dependent variable, subgrade type does not appear to have a significant effect on pavement distress.

Factors Affecting Pavement Cracking

A study of data given in Tables 4 and 5 indicates that

1. Spalled cracks are primarily induced by traffic; and
2. The extent of parallel cracks with a crack spacing less than 30 in. (76 cm) and random cracking observed per section of pavement are significantly influenced by traffic and the interaction between methods of paving and placing steel reinforcement.

Detailed Study of Factors Influencing Performance of CRC Pavements

Tables 6 and 7 give further elucidation of the results of analysis of variance given in Tables 2 through 5 and were developed on the basis of these results. These tables show the effect of excluding the data from 2 construction contracts. These contracts were treated separately because one of them developed distress shortly after it was opened to traffic and the other is the oldest CRCP contract (1964) included in this study.

Effect of Subgrade—The analyses indicate that subgrade parent materials had no significant effect on pavement distress or the extent of observed cracking.

Effect of Subbase—The results of data analysis show that subbase type has a major influence on pavement distress. Table 6 gives the effect of subbase on the distribution of defects per section. Bituminous-stabilized and crushed-stone subbases performed significantly better than gravel subbases. Slag subbases showed relatively poor performance. This conclusion needs a slight modification since all the defects related to slag subbases were confined to one construction contract.

Until the statewide condition survey, sections with bituminous-stabilized subbases did not show any significant distress and some sections with crushed-stone subbases showed minor distress. This conclusion should be viewed with caution as bituminous-stabilized subbases were used more recently (primarily 1972) and have not been exposed to the full range of environmental and traffic conditions. Since the time of the condition survey, severe distress has been reported on at least one contract with a bituminous-stabilized subbase.

The type and quality of subbases also have a significant influence on pavement pumping. Yoder (16) indicated that 3 basic conditions must be present to create pumping: frequent repetition of heavy loads, fine-grained material that will go into suspension with water, and free water under the pavement. The effect of subbase on pumping of CRC pavements is given in Table 8. Edge pumping was the primary mode of pavement pumping observed during the field survey, although pumping at cracks has been noted. The extent of observed pavement pumping was divided into 3 categories: no pumping; minor pumping, when pumping was indicated on less than 10 percent of the section

Table 3. Least squares analysis of variance of number of asphalt patches and breakups per section.

Source	DF	Sums of Squares	Mean Squares	F	$F_{0.05}$	Significant at $\alpha = 0.05$
Total (uncorrected)	95	73.363				
Main effects						
A_i	1	0.820	0.820	2.21	3.97	—
B_j	1	0.194	0.194	0.52	3.97	—
C_k	2	2.451	1.225	3.30	3.13	Yes
D_l	3	4.727	1.576	4.25	2.74	Yes
F_m	1	0.678	0.678	1.83	3.97	—
Interaction effects						
AB_{ij}	1	1.526	1.526	4.11	3.97	Yes
AC_{ik}	1	1.209	1.209	3.26	3.97	—
CF_{km}	2	2.013	1.007	2.71	3.13	—
Covariate effects						
S_{ijklmp}	1	1.389	1.389	3.74	3.97	—
T_{ijklmp}	1	1.647	1.647	4.44	3.97	Yes
$\epsilon_{(ijklm)p}$	80	29.670	0.371			

Table 4. Least squares analysis of variance of number of spalled cracks per section.

Source	DF	Sums of Squares	Mean Squares	F	$F_{0.05}$	Significant at $\alpha = 0.05$
Total (uncorrected)	95	147.099				
Main effects						
A_i	1	0.460	0.460	0.51	3.95	—
B_j	1	0.078	0.078	0.09	3.95	—
Interaction effects						
AB_{ij}	1	1.647	1.647	1.82	3.95	—
Covariate effects						
S_{ijklmp}	1	3.105	3.105	3.43	3.95	—
T_{ijklmp}	1	4.414	4.414	4.88	3.95	Yes
$\epsilon_{(ijklm)p}$	89	80.460	0.904			

Table 5. Least squares analysis of variance of length of pavement section showing random cracks plus parallel cracks.

Source	DF	Sums of Squares	Mean Squares	F	$F_{0.05}$	Significant at $\alpha = 0.05$
Total (uncorrected)	95	11 786 128.5				
Main effects						
A_i	1	27 013.3	27 013.3	0.33	3.98	—
B_j	1	43 741.2	43 741.2	0.53	3.98	—
C_k	2	7 983.0	3 991.5	0.05	3.12	—
D_l	3	119 550.5	39 850.2	0.48	2.74	—
F_m	1	201.1	201.1	0.002	3.98	—
Interaction effects						
AB_{ij}	1	458 089.4	458 089.4	5.51	3.98	Yes
BD_{jl}	2	239 911.7	119 955.9	1.44	3.12	—
BF_{jm}	1	198 573.4	198 573.4	2.39	3.98	—
CD_{kl}	5	838 289.3	167 657.9	2.02	2.35	—
Covariate effect						
T_{ijklmp}	1	317 479.5	317 479.5	3.82	3.98	—[a]
$\epsilon_{(ijklm)p}$	76	6 319 783.3	83 155.1			

[a]Significant at $\alpha = 0.10$.

Table 6. Effect of type of subbase on distribution of average number of defects per section.

Subbase	Number of Sections	Concrete Patches, Asphalt Patches, and Breakups	Asphalt Patches and Breakups	Concrete Patches
Gravel	46	1.42	1.08	0.34
	44[a]	0.80	0.74	0.06
Slag[b]	8	0.75	0.62	0.13
Crushed stone	20	0.15	0.15	0.00
Bituminous stabilized	15	0.00	0.00	0.00

[a]Excluding 2 construction contracts.
[b]All defects observed on 1 construction contract.

length; and major pumping, when pumping was indicated on more than 10 percent of the section length.

Data given in Table 8 show that the highest incidence of pumping occurred where gravel subbases were used; no pumping was indicated on sections with slag subbases. Minor pumping was observed on sections with crushed-stone and bituminous-stabilized subbases.

Effect of Type of Steel Reinforcement—Table 7 gives the distribution of average number of defects, average number of spalled cracks, and average length of cracking observed per pavement section for various combinations of construction factors. Conclusions are as follows:

1. Use of bar mats resulted in more defects per section than use of wire fabric or loose bars (this statement should be qualified by the fact that bar mats were used mainly in older CRCP contracts and loose bars were used more recently, primarily 1972);
2. For various combinations of methods of paving and steel placement, use of wire fabric resulted in more widespread cracking; and
3. The use of wire fabric resulted in relatively more distress in slip-formed pavement sections than in side-formed sections.

Effect of Method of Steel Placement—For various combinations of steel type and paving method, a larger number of defects per section were observed when chairs were used as a method of placement (Table 7). This relation breaks down in the case of a combination of a side-formed pavement and bar mats. Here the use of a depressor resulted in a relatively larger number of defects. More cracking was also evidenced in sections where chairs were used for placing steel reinforcement. This relation does not hold for the case of a side-formed pavement reinforced with wire fabric. In this case, use of a depressor resulted in greater amount of cracking.

Effect of Method of Paving—By itself, the method of paving has no significant effect on pavement distress (Table 3). The incidence of cracking was relatively greater in side-formed pavements than in slip-formed pavements for various combinations of steel reinforcement and method of steel placement.

Effect of Traffic—The time that a pavement has been under traffic has a significant effect on pavement distress.

Effect of Concrete Slump—Table 9 gives the effect of slump on the distribution of defects. A higher percentage of sections had defects where the concrete slump was low. The optimum value of slump relative to performance is between 2.0 and 2.5 in. (5.0 to 6.4 cm). With increase in slump, a decrease in the number of defects per section is also indicated. The effect of slump values, greater than 2.5 in. (6.4 cm), on the occurrence of defects should be carefully considered. There were only 6 sections having slump values greater than 2.5 in. (6.4 cm), and these may not be representative of the effect.

Distribution of Defects

Figure 4 shows the frequency distribution of defects observed on 89 sections, each 5,000 ft (1524 m) long, of equivalent 2- or 3-lane CRC pavement.

CONCLUSIONS

Based on a statistical analysis of data collected during a statewide survey of continuously reinforced concrete pavements in Indiana, the following conclusions are presented. The survey was statistically designed wherein each construction contract was required to be in the study. At least 1 survey section 5,000 ft (1524 m) in length was sampled from each contract. In some cases, more than one 5,000-ft (1524-m) section was observed within a construction contract because of the stratification of factors used in the statistical study.

The results of the statewide survey have given some definite indications relative to causes of distress in CRC pavements as will be pointed out below. However, several questions remain unanswered concerning the reasons for distress on certain CRCP sections in the state. In view of this, a continuing field and laboratory study of CRC pavements is currently in progress at Purdue University.

Table 7. Effect of method of construction on distribution of average number of defects and spalled cracks and average length of cracking.

Paving	Number of Sections	Concrete Patches, Asphalt Patches, and Breakups	Asphalt Patches and Breakups	Length of Cracking[a] (ft)	Spalled Cracks	Years of Traffic
Slip-formed						
Loose bars						
Depressor	4	0.0	0.0	191.5	0.25	1.04
Chairs	13[b]	0.15	0.15	464.2	0.77	0.61
	14	1.5	0.64	478.8	1.14	0.63
Wire fabric						
Depressor	14	0.53	0.53	308.3	0.43	0.54
Chairs	11	0.64	0.55	596.8	3.82	1.62
Bar mats						
Depressor	12	0.67	0.58	191.1	0.92	0.41
Chairs	13	1.08	1.0	543.5	2.89	2.21
Side-formed						
Wire fabric						
Depressor	12	0.08	0.08	669.4	2.46	1.97
Chairs	4[c]	0.25	0.25	383.5	2.75	1.62
	5	2.4	2.2	508.0	6.40	2.92
Bar mats						
Depressor	2	1.0	1.0	412.0	0.00	0.25
Chairs	2	0.50	0.50	866.0	0.67	3.25

[a] Longitudinal pavement length showing parallel cracks less than 30-in. (76-cm) spacing plus longitudinal pavement length showing random cracking.
[b] Exluding the contract that showed immediate distress.
[c] Exluding the oldest CRCP contract.

Table 8. Effect of subbase type on amount of pumping in CRCP sections.

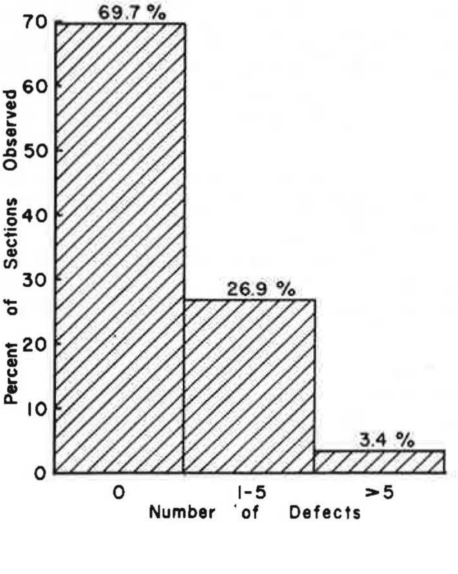
Subbase	Number of Sections	No Pumping		Minor Pumping		Major Pumping	
		Percent	Number	Percent	Number	Percent	Number
Gravel	46	56.5	26	34.7	16	8.8	4
Crushed stone	20	90.0	18	10.0	2	-	-
Bituminous stabilized	15	93.3	14	6.7	1	-	-
Slag	8	100.0	8	-	-	-	-

Table 9. Effect of slump on distribution of defects among sections.

Slump (in.)	Number of Section Surveyed	Sections With No Defects	Sections With Defects		
			Number	Percent	Number per Section[a]
1.0-1.5	31	17	14	42.2	2.9
1.5-2.0	39	29	10	25.6	3.2
2.0-2.5	13	11	2	15.4	1.0
2.5-3.0	4	3	1	25.0	1.0
>3.0	2	2	0	0.0	0.0

Note: 1 in. = 2.5 cm.
[a] Only section with defects considered.

Figure 4. Frequency distribution of defects observed in the statewide CRCP survey.



The following conclusions pertain to the effect of various factors influencing the performance of CRC pavements in Indiana.

1. Subbase type was found to be a significant contributor to the performance of CRC pavements; gravel subbases showed the poorest performance. Crushed stone and slag subbases have, in general, shown good performance, and until the survey the bituminous-stabilized subbases showed little or no distress. However, since the survey was conducted, some breakup has been encountered on at least one bituminous-stabilized subbase.
2. Depressed steel performed significantly better than preset steel used on chairs.
3. All other factors being constant, loose bars and welded wire fabric showed good performance. Bar mats showed the poorest performance, but this type of steel was used mainly on some of the earlier projects and, thus, these pavements have been exposed to a wider range of environmental and traffic conditions.
4. Concrete slump has a significant effect on pavement performance; the optimum slump range was between 2.0 and 2.5 in. (5.0 to 6.4 cm). Slump values of 1.5 in. (3.8 cm) and greater have shown good results.
5. Pavements that were side-formed performed the same as those that were slip-formed.
6. Much of the distress takes place during the cold months of the year, suggesting that extreme temperature drops have a major effect on performance.
7. Distress of CRC pavements is associated with traffic which apparently is on the increase in Indiana.
8. The primary mode of pumping of CRC pavements is edge pumping. The results of the condition survey indicate that pavements with gravel subbases are more susceptible to pumping. Pavements with crushed-stone and bituminous-stabilized subbases have shown some indication of pumping; pavements with slag subbases have not pumped.
9. Subgrade parent material type (granular or fine-grained) was not a significant contributor to performance of CRC pavements. This refers to type of subgrade and not to other factors such as degree of compaction.

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EXPERIENCE WITH CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS IN VIRGINIA

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This paper reports studies of the cracking characteristics of continuously reinforced concrete pavement (CRCP) built in Virginia since 1966 and observations concerning the performance of this type of pavement. Early studies involved crack frequency, in situ width determinations, and laboratory examinations of cores. Later studies determined the seasonal movement of cracks and the patterns of crack development with time. All studies were intended to determine the normalcy of characteristics of Virginia CRC pavement as compared to characteristics reported by other agencies. It is concluded from the studies that cracking patterns tend to be fully developed in 2 to 3 years and that an early progressive increase in surface crack widths seems to stabilize after a similar period of time. The core studies showed that cracks are discontinuous in the immediate vicinity of the reinforcing steel and grow progressively wider toward the top and bottom surfaces. This finding is taken as evidence that no danger of steel corrosion exists in pavements having normal crack patterns. The performance of the Virginia CRC pavement is considered good. However, at least 1 project shows severe damage related to inadequate consolidation of the paving concrete. The need for a strong effort toward the development of realistic consolidation specifications is pointed out.

•THE first continuously reinforced concrete pavement (CRCP), about 15 miles (24 km) in length, was built in Virginia in late 1966 and early 1967 on Interstate 64 around Richmond. Later projects brought the total to some 185 miles (298 km) by the end of 1972. All projects have been constructed in moderately heavy traffic corridors on the Interstate system.

The earlier pavements were constructed on unstabilized subgrades and subbases and contained both longitudinal and full transverse reinforcement. Later pavements were constructed on cement-stabilized subgrades and subbases and contained only longitudinal reinforcement with transverse tie bars between 12-ft (3.66-m) lanes. Similarly, there has been a transition from the use of side forms to slip forms in the placement operation.

A summary of the existing design features of CRCP in the state is given in Table 1. The pavements are 8-in. (20-cm) thick with 0.6 percent longitudinal reinforcement. Periodically, studies have been conducted in an effort to answer some of the questions that arose as changes were made in design features. Are the cracks in continuously reinforced concrete pavements in Virginia different in frequency from those found on such pavements in other states? Are the crack widths different from those found in other states? Are the cracks wide enough to allow corrosion of the reinforcement? Are cracking frequency and width related to some important variables in construction? Do the cracks tend to grow wider and more frequent with age? What are the effects of the elimination of full width transverse reinforcement? Has a cement-stabilized subbase led to a substantially better pavement structure?

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The first 4 questions were dealt with in reports to the Virginia Highway Department (1, 2), and the last 3 questions are still under consideration in a study on which 1 progress report has been issued (3). This paper summarizes the findings of these studies through early 1973. Several noteworthy performance problems also are discussed.

CRACKING FREQUENCY

Cracking frequency and its changes have been determined by periodic visual examination of a number of randomly selected, 200-ft (61-m) long test sites on projects given in Table 1 as groups A and D.

Group A Pavements

The results of 2 surveys of cracking frequency on several of the group A projects on I-64 near Richmond are given in Tables 2 and 3. The spacings shown are averages for two 12-ft (3.66-m) lanes unless otherwise indicated. Exact locations, paving dates, and climatic conditions at the time of paving are given in another report (1).

For the western sites (Table 2) there was a general reduction in crack spacing between 1967 and 1969. On the other hand, the section having the shortest spacing in 1967 (site 4, westbound) remained unchanged in 1969, which suggests that the crack pattern here was already fully developed at the time of the earlier survey. The average spacing for all western sites was 3.88 ft (1.18 m) in 1969 and to 4.70 ft (1.40 m) in 1967. These averages do not reflect the fact that some cracks are spaced 6 in. (0.15 m) to 1 ft (0.30 m). Such averages are consistent with experiences in other states (4), and the agreement of the averages with those from other states was considered important.

The 1969 survey shows that, unlike in 1967, the western and eastern test sections now are remarkably similar in crack spacings [average of 3.83 ft (1.17 m) and 3.97 ft (1.21 m) for west and east respectively]. This finding indicates that the crack pattern in the eastern sections was not so fully developed in 1967 as that in the western sections. This was no doubt due to the fact that most of the eastern sections were placed in cool weather (fall 1966 and spring 1967) and were surveyed before much exposure to extreme temperature changes. The western sections, on the other hand, were all placed in the summer of 1966 and had been subjected to extreme temperature changes before the 1967 survey. Time and cyclic temperature extremes by 1969 appear to have obscured any differences present in the earlier survey.

Group D Pavements

The results of similar, but later, cracking-frequency surveys on several group D (no transverse steel) pavements are given in Table 4. In this case, the average crack spacings have been determined monthly for each of the eleven 200-ft (61-m) long test sections during the past 3½ years. Precise site locations and other pavement details are given in another report (3).

The spacing for the spring and summer concrete is approaching that expected for the type of pavement under study. Similar pavements (0.6 percent longitudinal reinforcement) had crack spacings of 3 to 5 ft (0.91 to 1.52 m) at ages of 2 to 3 years.

The winter concrete, on the other hand, shows signs that the spacing may never be so close as expected. This is no doubt due to the extremely cold weather during which the concrete was placed. Overnight temperatures typically fell below 30 F (-6 to -2 C) during this December 1969 paving period. The nearly 13-ft (3.96-m) average crack spacing contrasts sharply with that for some of the July 1970 pavement, which had an average spacing of about 15 ft (4.57 m) within 48 hours after placement at temperatures ranging to 90 F (32 C). Clearly, hydrothermal stresses have had a more pronounced effect on the summer than on the winter concrete. It is apparent that the winter concrete, placed at very low temperatures, has seldom been exposed to significant tensile stresses and that, unless the pavement experiences an extremely cold and dry winter, the number of cracks may never increase significantly.

CRACK WIDTHS IN GROUP A PAVEMENTS

Crack Measurements In Situ

The width of each tenth crack within the 200-ft (61-m) sites was determined in each wheelpath by means of a measuring microscope at a magnification of about 15x. This method is subject to greater operator interpretations than is a mechanical strain gauges measuring between reference points but has been shown to be reliable (1).

The optics are such that the measurement is made at some depth below the surface [certainly less than $\frac{1}{16}$ in. (0.16 cm)]. At this point there are usually 2 definite "edges" of the crack. The microscopic method thus gives the width near the surface, which is probably somewhat greater than comparable widths measured with mechanical gauges from reference points.

The average crack widths for each of the sites west of Richmond are given in Table 5. The averages represent measurements at right and left wheelpaths on 3 cracks for each traffic lane, so that each average is a composite of 6 individual measurements. Crack widths were measured only in the eastbound lane.

There is little difference between the 1967 and 1969 crack widths for the western projects. Some sites indicate a slight closing of the cracks, others a slight opening. These minor differences may be due to the inherent difficulty in reproducing exact measurements on the very variable crack widths. The average widths of all cracks for 1967 and 1969 [0.012 in. (0.031 cm) and 0.014 in. (0.036 cm) respectively] are very close and probably indicate no true change. Average temperatures were 56 F (13 C) in 1967 and 40 F (4.4 C) in 1969.

Crack-width measurements on the eastern projects are given in Table 6. In these cases, crack measurements were randomly distributed between the eastbound and westbound lanes. The 1969 measurements were approximately double those for 1967. Although this increase in crack widths may be partially due to a more complete development of the crack pattern, as indicated by the crack spacings on these sections, it is also doubtlessly influenced by the marked difference in temperatures at the time of the 2 surveys, 80 F (27 C) in 1967 and 40 F (4.4 C) in 1969. The higher temperature in 1967 would have tended to hold the cracks tighter. In any event, the effect of the larger crack widths for these eastern sites is to make the eastern and western sites more similar, whereas in 1967 considerable differences existed between the two. In 1969, the average crack width for the 2 areas was the same, 0.014 in. (0.036 cm); the 1967 average for the eastern sites was 0.006 in. (0.015 cm).

Again, as with crack spacing, the crack-width data compared well with those from similar pavements reported by other states (4) so that the group A pavements were considered to be performing satisfactorily.

Crack Measurements on Cores

The possibility of corrosion of steel depends primarily on the width of crack at the steel, which in turn depends on the width of the surface crack and the depth of cover of concrete over the steel. Suggested maximum limits vary, but a recent survey of pertinent literature published at the University of Illinois (5) indicates that a limit on surface crack width of about 0.006 to 0.010 in. (0.015 to 0.025 cm) would be appropriate for members in aggressive environments but with a 1 to 2 in. (2.5 to 5.0 cm) cover. This report further states:

> If the crack is designated by its magnitude at the concrete surface, then depth of cover will control the limit on crack width. A limiting crack width so defined will increase with an increase in cover simply because the corresponding dimension of the crack at the reinforcement will be smaller.
>
> This would mean that for depths of cover of 3½ to 4 in. (9 to 10 cm) the allowable crack width at the surface would be about 0.0200 in. (0.051 cm).

To gain additional information concerning the possibility of reinforcement corrosion, we took a 4-in. (10-cm) diameter core from the pavement at each site in the western

Table 1. Continuously reinforced concrete pavement design features.

Group	Subgrade	Subbase	Transverse Reinforcement	Placement Method	Year Built	Mileage
A	Select material	Aggregate base	No. 4, 30 in., c. to c.	Side form	1966-67	26.98
B	Native	Cement-treated select material, min CBR 30	No. 4, 30 in., c. to c.	Side form	1968-69	13.92
C	Native	Cement-treated aggregate base	No. 4, 30 in., c. to c.	Slip form	1968-69	44.52
D	Soil-cement	Cement-treated aggregate base	Tie bars only	Slip form	1969-71	55.74
E	Soil-cement	Cement-treated aggregate base	Tie bars only	Slip form	1971-73	43.80

Note: 1 in. = 2.5 cm; 1 mile = 1.6 km.

Table 2. Average crack spacings for group A projects west of Richmond.

Site	Eastbound (ft) 1967	Eastbound (ft) 1969	Westbound (ft) 1967	Westbound (ft) 1969
1	4.35	3.85	4.90	3.79
2	4.17	3.38	4.41	4.00
3	4.51	4.55	4.60	4.45
4	3.04	3.03	4.30	4.22
5	5.30	3.68	5.01	4.09
6[a]	5.60	3.37	6.10	4.17
Avg	4.50	3.64	4.89	4.12

Note: 1 ft = 0.3 m.
[a]Three 12-ft (3.66-m) lanes.

Table 3. Average crack spacings for group A projects east of Richmond.

Site	Eastbound (ft) 1967	Eastbound (ft) 1969	Westbound (ft) 1967	Westbound (ft) 1969
1	6.17	3.11	5.60	3.42
2	10.66	4.43	7.44	3.92
3	10.64	4.20	7.18	3.83
4	68.75	3.76	12.03	4.51
5	32.67	4.13	7.59	4.51
Avg[a]	25.78	3.92	7.97	4.03

Note: 1 ft = 0.3 m.
[a]Avg is 16.87 ft for 1967 and 3.97 ft for 1969.

Table 4. Average crack spacings for group D pavements.

Date Paved	After 2 Months	After 1 Winter	After 2 Winters	After 3 Winters
May 1970	9.09	5.37	5.10	5.00
July 1970	8.57	5.97	4.92	4.96
December 1969	20.51	13.33	12.90	12.50

Note: Spacings are in feet, where 1 ft = 0.3 m.

Table 5. Average crack widths for projects west of Richmond in eastbound lane.

Site	Traffic Lane (in.) 1967	Traffic Lane (in.) 1969	Middle Lane (in.) 1967	Middle Lane (in.) 1969	Passing Lane (in.) 1967	Passing Lane (in.) 1969
1	0.012	0.019	–	–	0.012	0.011
2	0.013	0.015	–	–	0.015	0.013
3	0.012	0.010	–	–	0.009	0.008
4	0.014	0.019	–	–	0.013	0.011
5	–	0.022	–	–	–	0.018
6	0.009	0.016	0.012	0.013	0.016	0.011
Avg	0.012	0.017	0.012	0.013	0.013	0.012

Note: 1 in. = 2.5 cm.

Table 6. Average crack widths for projects east of Richmond.

Site	Direction	Traffic Lane (in.) 1967	Traffic Lane (in.) 1969	Middle Lane (in.) 1967	Middle Lane (in.) 1969	Passing Lane (in.) 1967	Passing Lane (in.) 1969
1	East	0.006	0.015	0.005	0.018	0.005	0.017
2	West	0.009	0.013	0.006	0.016	0.005	0.008
3	West	0.008	0.014	0.005	0.016	0.005	0.010
4	East	0.001	0.010	0.001	0.010	0.006	0.015
5	East	0.005	0.018	0.002	0.018	0.013	0.015

Note: 1 in. = 2.5 cm.

area for which cracking data had been collected. Of major concern in these cases was the nature of the cracks at points closer to the steel than could be studied in the field.

In the laboratory, photographs were taken of the top and each side of each core. The same measuring microscope that was used in the field work was used to determine the crack width at the surface of each core. Then the cores were sawed horizontally at distances of first 1½ in. (3.8 cm) and then ½ in. (1.3 cm) above and below the longitudinal steel, and crack measurements were made at these levels after the cut surfaces had been polished. Photographs illustrating the appearance of a typical crack at the various surfaces are shown in Figures 1, 2, 3, 4, and 5. Finally, the measurement of horizontal and vertical components permitted the calculation of the straight-line distance from the surface of the main reinforcing steel to the point of crack measurement.

The results of crack measurements for 5 of the cores are shown in Figure 6. For the measured cracks, the width tends to decrease by approximately a factor of 2 as the distance from the surface of the steel changes from 4½ to ¾ in. (11.4 to 1.9 cm). Although, as the scatter of points in Figure 6 shows, the correlation between crack width and distance from the steel is not high, definite trends can be seen in both the figure and in the visual examination of the cores. Projecting the crack measurements shown in Figure 6 to the surface of the steel suggests that at the steel a crack of approximately 0.003 in. (0.008 cm) still would exist. Visual examination (Fig. 5) showed that this was not the case and that no definite crack exists directly at the steel. Such a result has been reported in another paper (6) in which it was noted that in some cases cracks tend to diverge in the neighborhood of the steel and that it has been possible to find a zone around the steel where the concrete is not really cracked but is distended in a discontinuous manner. Such appears to have been the case with the cores in question. Figures 4 and 5 show that the cracks extend through the entire depth of the pavement but are discontinuous in the vicinity of the steel. Measurements indicated that whether cracks are located above or below the steel has little influence on the crack width and that the proximity of steel to the point of measurement is the determining factor.

The crack widths measured at the surface are of the same order of magnitude as those on pavements performing satisfactorily in other states, and the vast majority are within recommended limits for structural members if allowance is made for depth of concrete cover and age. Furthermore, the laboratory studies show that no definite crack exists at the surface of the steel for any of the cores examined. Thus, it is concluded that the cracks present no threat of steel corrosion and that the steel is functioning as designed, i.e., to restrain the concrete such that fine, harmless cracks develop at relatively close intervals.

Cores removed subsequent to the above studies have shown no evidence of steel corrosion except in cases were honeycombing has left the steel more vulnerable.

CRACK WIDTHS IN GROUP D PAVEMENTS

Because of the hazardous situation created by attempting to measure crack openings on Interstate highways with the measuring microscope used on the group A pavements, the changes in crack widths for the group D pavements are being measured by the use of gauge plugs spanning randomly located cracks within 11 test sections.

The gauge plugs are ½-in. (1.3-cm) diameter by 1-in. (2.5 cm) long brass cylinders tightly bound in predrilled holes with a paste made of expansive cement. Gauge points were drilled in the plugs after several days were allowed for the cement paste to harden. A nominal gauge length of 10 in. (25.4 cm) is used for monthly measurements with a dial gauge. Initial measurements were made with a Whittemore gauge, which was later replaced by a gauge with a larger dail (Fig. 7) built in the Virginia Research Council shop. The latter gauge was necessary because end anchorage movement (not reported here) exceeded the capacity of the Whittemore gauge. Measurements are recorded to the nearest 0.0001 in. (0.0003 cm) and are corrected for concrete volume changes within the gauge length. This correction is computed from measurements made on uncracked pavement immediately adjacent to the cracks. Average crack widths for the 3 study groups at critical times (maximum and minimum seasonal

Figure 1. Top view of core 6.

Figure 2. Plan view of core 6 at sawed surface 1½ in. (3.8 cm) above steel.

Figure 3. Plan view of core 6 at sawed surface 1½ in. (3.8 cm) below steel.

Figure 4. Left side view of core 6 showing continuous crack through core.

Figure 5. Right side view of core 6 showing discontinuous crack in vicinity of steel.

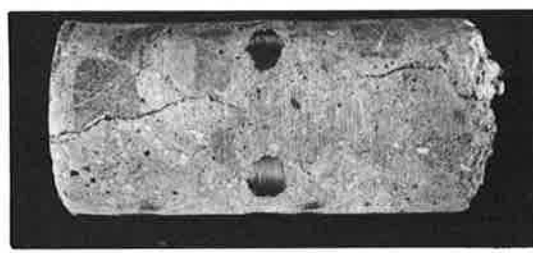
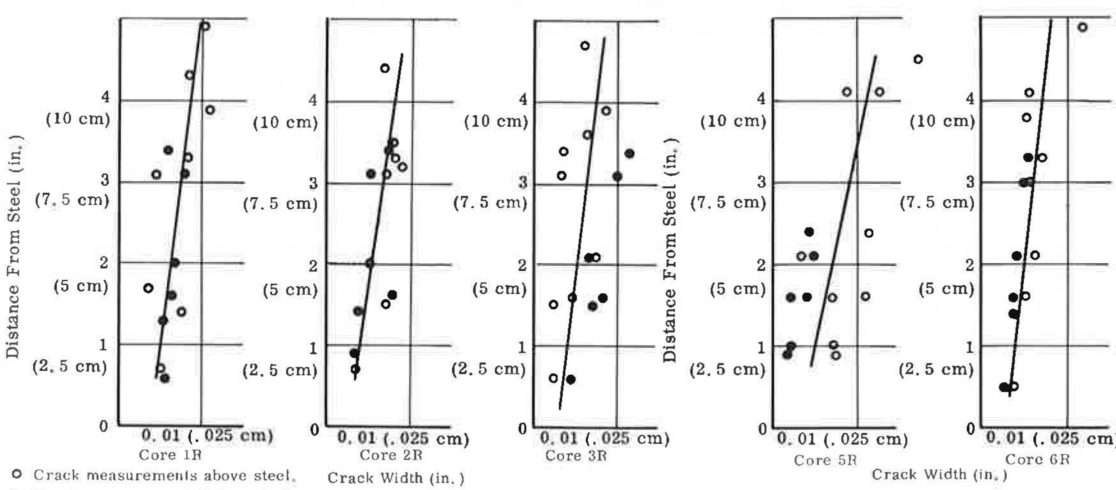


Figure 6. Crack width versus distance from steel for 5 cores.



measurements) are given in Table 7, where each entry represents an average of 12 measurements. Cracks were originally instrumented at an early date and at a width assumed to be nearly zero.

There is no apparent relation between paving date, crack spacing, and crack width. There is a pronounced seasonal effect as evidenced by the differences between the summer (narrow) and the winter (wide) openings. The summer 1972 measurements are wider than those for the summer of 1971. This may indicate a gradual increase in width. On the other hand, it may partially result from different seasonal conditions between the 2 summers. Further studies during the next several years should determine whether there is a progressive increase in crack width. The 1971-73 winter openings of 0.01 to 0.03 in. (0.03 to 0.09 cm) are in line with the results of earlier studies both in Virginia and in other states for similar pavements (1, 2).

EFFECTS OF THE ELIMINATION OF TRANSVERSE STEEL

Periodic examinations of the group D and E projects along with a study of the relevant literature have revealed no detrimental effects of the elimination of transverse steel from continuously reinforced concrete pavements. Although there was at one time some concern that occasional random longitudinal cracking was related to the elimination of the steel, studies showed that the cracking seldom went beyond the limits of the transverse tie bars provided between the 12-ft (3.66-m) lanes. It was also determined that several older projects having full-width transverse steel had even more of the longitudinal cracking than the projects from which the steel was eliminated.

The conclusion that elimination of the steel has not influenced the behavior of "no transverse steel" pavements is further substantiated by the similarity of their cracking characteristics, discussed earlier, to those on the group A projects studied in 1967 and 1969.

STRUCTURAL COMPARISON OF PAVEMENTS WITH STABILIZED AND UNSTABILIZED LOWER LAYERS

Although no extensive structural studies have been conducted, a series of Benkelman beam deflection tests was run recently on two 1,500-ft (457-m) long segments of pavement from group A (no stabilization) and group E (stabilized subgrade and subbase).

Edge deflections were measured at 50-ft (15.2-m) intervals through the test sections. Each measurement was made just as a dump truck loaded to 18,000 lb (8165 kg) on the rear axle traveled the normal wheelpaths past the point of testing.

The CRCP edge deflections (in inches, where 1 in = 2.5 cm) were as follows:

Group	Min	Max	Avg
A	0.008	0.016	0.010
E	0.004	0.010	0.006

The addition of the stabilized layers has been accompanied by a 40 percent average reduction in deflections. Although this reduction may be altogether due to the stabilization, slightly better subgrade soils in the stabilized area may have been a contributing factor. It is hoped that these preliminary deflection tests can be supplemented by tests on each pavement group.

PERFORMANCE PROBLEMS

Although the CRC pavement in Virginia has generally performed as expected, there have been several noteworthy problems that are indicated below in the order of their severity:

1. Pavement failures due to poor consolidation of concrete,
2. Pavement failures due to poor quality concrete,
3. Loss of shoulder material due to edge pumping related to inadequate drainage, and

4. Random longitudinal cracking.

A brief discussion of each of these performance problems follows.

Failures Due to Poor Consolidation

The most severe performance problem to occur in the CRCP to date is a result of honeycombing caused by inadequate consolidation of the paving concrete. An example of the most serious failure is shown in Figures 8 and 9. Figure 8 shows a segment of 8-in. (20-cm) thick by 24-ft (7.31-m) wide slip-formed CRCP from the group C pavements. A temporary bituminous concrete patch is evident in the picture. This type of failure is characterized by wide, arrow-straight cracks located directly above the transverse steel and by edge pumping, also shown in Figure 8.

The cause of this type of failure was immediately revealed by full-depth cores taken through the transverse cracks and at intermediate points between cracks. Figure 9 shows a side view of one of the cores taken through a crack at the transverse steel. There is a noticeable lack of consolidation of the concrete directly below the steel. The result is a void directly under the steel with a very wide crack extending from the void downward to the bottom of the CRCP. No doubt the reduced cross section caused by the void under the steel has resulted in the wider than normal crack at the pavement surface and in the crack lying in the same vertical plane as the steel. Strangely, cores taken from between the transverse bars showed no visual evidence of poor consolidation. Edge pumping in the section appears to have resulted from surface water entering the wide transverse cracks and becoming trapped between the pavement and the cement-stabilized subbase. The stabilization has had the advantage of preventing the loss of significant amounts of subbase material.

This type of distress has progressed to a serious degree in several segments on one contract in the state. All paving was with a slip-form paver employing "spud" vibrators at an unknown spacing and frequency. Also, there are no records of paver speed through the failed sections.

A second type of consolidation problem has occurred at construction joints and has been found to at least a small degree on almost all CRC pavements built in the state. These failures are much less severe than the type described above and are characterized by wide cracks at construction joints and by nearby potholes that are underlaid by honeycombed areas. These failures have emphasized the need for greater attention to vibration at the construction joints, where consolidation is more difficult because the longitudinal steel is doubled. The repair of problems caused by poor consolidation has been time-consuming and costly. In most cases the longitudinal steel is left in place, and new concrete is placed around it. However, the time consumed in chipping the old concrete from the steel has led to long lane closures and high traffic control costs.

The consolidation problems encountered in Virginia, although severe on only one contract, serve to point up the already recognized need for realistic vibration specifications for paving concrete.

Failures Due to Poor Quality Concrete

Several failures on one of the earlier (group A) CRCP projects built in the state resulted from poor quality concrete caused by excessive water-cement ratios. These failures are few in number and usually involve only a few square yards of pavement. The failures are characterized at early ages by closely spaced transverse cracks that are joined by short longitudinal cracks. Deep surface erosion has resulted from relatively few vehicle passes even without the application of de-icing chemicals. To date, none of these areas has been replaced, but several have small temporary bituminous patches.

Since the failed areas tend to represent about 1 batch of paving concrete, it was concluded that poor control of the concrete placed by a drum road mixer was to blame. All subsequent CRC pavement has been from stationary central mix plants, and the problem has not been detected on these projects.

Edge Pumping Due to Poor Drainage

An example of edge pumping due to poor drainage is shown in Figure 10, where staining of the bituminous shoulder is evident. The problem has occurred on the group E projects where the subbase is cement stabilized and the shoulder base material is unstabilized sand and gravel with a 2 to 3 plasticity index. The condition exists primarily at the low point of sag verticals.

Trenching and edge deflection tests showed that no measurable void existed under the CRCP. It was concluded that the pumping resulted from surface water trapped at the interface between the CRCP and the impervious shoulder material. The stains were attributed to fines lost from shoulder material, as was further evidenced by a slight depression of the shoulder at its contact with the CRCP.

Corrective action is to take the form of trenching the shoulder at the pavement edge and backfilling with a highly pervious 1-sized aggregate. Water will be carried to the ditch lines with occasional side drains filled with the same pervious crushed stone.

Design standards are being revised to prevent recurrence of the problem by the provision of nonplastic, pervious shoulder material.

Random Longitudinal Cracking

Since the introduction of slip-form paving using the polyethelene strip to create a weakened plane longitudinal joint, some of the CRC pavement has suffered a modest amount of random longitudinal cracking (group C and D pavements). This experience has been reported by many other agencies using similar construction procedures.

Fortunately, in Virginia the polyethelene tape and cement stabilization of subbases occurred simultaneously so that there is little danger, and no evidence, of faulting of the longitudinal cracks. Another factor making both faulting and the development of excessive crack widths unlikely is the tendency of the random cracks to occur within the limits of the 30-in. (76-cm) long tie bars spanning the center joint.

For these reasons, random longitudinal cracking is not considered to be a serious performance problem in Virginia at this time.

Fill Settlements

Although fill settlements are not strictly a pavement problem, several of the CRCP projects built in the state have been subject to distortions due to such settlements, particularly those directly over minor drainage structures. The structural integrity of the pavements does not seem to be affected by these deformations unless unsound concrete is present. Thus, in almost all cases the pavements conform to the fill settlements, and there is no evidence of the development of unusual cracks.

Such conditions are routinely corrected by mudjacking, and the pavements return to grade with no evident defects. As far as can be determined, the only way to combat these settlements is through careful attention to backfill and embankment construction.

CONCLUSIONS

The following conclusions appear to be warranted from the findings presented in this paper. Some of the pavements are relatively young, and therefore some of the behavior patterns may not have fully developed.

1. CRCP has performed well in Virginia except in cases where design or construction technology has been deficient. These deficiencies have been most noticeable in the areas of pavement consolidation, quality control of concrete, and provision of adequate drainage systems. The need for a strong research effort in the development of consolidation specifications is evident.
2. Cracks show a definite trend to become more closely spaced and to develop greater surface widths during the first 2 to 3 years of the pavement's life. There is some inconclusive evidence that the widening process stabilizes after this time period.
3. Although crack spacing can be influenced by weather conditions during pavement placement, there is no apparent relation between crack width and either crack spacing

Table 7. Changes in crack width from 1970 to 1973.

Paving Date	Summer 1970	Winter 1970-71	Summer 1971	Winter 1971-72	Summer 1972	Winter 1972-73	Summer 1973
May 1970	0	0.0138	0.0034	0.0139	0.0073	0.0147	0.0105
July 1970	0	0.0170	0.0139	0.0305	0.0248	0.0332	0.0186
December 1969	0	0.0185	0.0068	0.0239	0.0160	0.0398	0.0183

Note: Amounts are in inches. 1 in. = 2.5 cm.

Figure 7. Extensometer and standard used for crack measurements.

Figure 8. CRCP failure due to poor consolidation.

Figure 9. Core taken from a poorly consolidated pavement.

Figure 10. CRCP edge pumping due to poor drainage of shoulder material.

or placement weather. Crack spacing seems to be markedly influenced by weather conditions only in the case of extremely low placement temperatures.

4. Crack widths measured at the pavement surface are of the same order of magnitude as those on pavements performing satisfactorily in other states, and the vast majority are within recommended limits for structural members if allowance is made for depth of concrete cover and age. Furthermore, laboratory studies show that no definite crack exists at the surface of the steel for any of several cores examined. Thus, it is concluded that the cracks present no threat of steel corrosion and that the steel is functioning as designed, i.e., to restrain the concrete such that fine, harmless cracks develop at relatively close intervals.

5. The elimination of full-width transverse steel has had no effect on pavement performance for the first 2 or 3 years.

6. Although there is no confirmed effect on performance at this time, deflection tests indicate that cement stabilization of subbases has markedly strengthened pavements built in Virginia. Stabilization may have prevented CRCP damage due to edge pumping and random longitudinal cracking.

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CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS IN MISSISSIPPI

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This report covers a 5-year observation study of continuously reinforced concrete pavement construction in Mississippi. The current design and construction practice for CRCP is reviewed, and special items such as splices, transverse reinforcement, terminal treatments, end construction arrangement, vibration of concrete, curing of concrete, and longitudinal center joint are discussed. Field measurements on present serviceability index, crack spacing, and deflection are included and evaluated. Also included is a proposed maintenance procedure for CRC pavements.

•MISSISSIPPI ranks third in the nation in mileage of continuously reinforced concrete pavement (CRCP). Almost 900 miles (1448 km) of 2-lane pavement are in use in the state. On a nationwide basis, more than 10,000 miles (16 000 km) of 2-lane pavement were in use or under contract at the end of 1971. Thirty-three states currently have some CRCP (7); 19 have 100 miles (160 km) or more including 2 states that have more than 1,000 miles (1600 km). The use of CRCP is increasing rapidly. Therefore, it is important that highway engineers have a complete knowledge of the design, construction, performance, and maintenance of this type of pavement.

This report summarizes the experience with CRCP in Mississippi. It covers the results of a 5-year observation study that included the design, construction, testing and maintenance of CRCP.

CURRENT DESIGN AND CONSTRUCTION PRACTICES

Current Practices in the Design of CRCP

Unlike that of most other states, the standard CRCP design procedure in Mississippi was primarily based on the research findings of the state's early experimental projects constructed in Desoto and Jones Counties (10, 17, 18). A report by the American Concrete Institute (1) and the Minimum Criteria for Federal-Aid Roads established by the then Bureau of Public Roads also played an important role in these standard design procedures.

Stabilized base and subbase are used in Mississippi under all CRC pavement. Design is based on the CBR value and the anticipated traffic load; usually the design of 6-in. (152-mm) soil-cement-treated base and 6-in. (152-mm) lime-treated subbase (may vary as required by soil condition) is used (12). Beginning in 1971, designs of 4-in. (102-mm) asphalt concrete base and 6-in. (152-mm) granular subbase or 6-in. (152-mm) lime-treated subbase (as required by soil condition) have been tried on several projects.

The standard thickness for CRCP in Mississippi is 8 in. (203 mm). Strength for the concrete is not specified. However, general practice in the field is that the concrete should provide a modulus of rupture of 525 to 550 lb/in.^{2} (3.6 to 3.8 MPa) at 7 days and 700 psi (4.8 MPa) or more at 28 days when tested by the third-point method.

The amount of longitudinal steel is 0.6 percent. Only deformed bars (with 60,000

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lb/in.^{2} or 414 MPa yield strength) are allowed. Smooth wire fabric was tried on 2 projects during the early 1960s and has since been discontinued as one of the CRCP design alternates. The current design uses No. 5 bars at 6½-in. (165-mm) spacing and has 0.037 in.^{2} of bond area per 1 in.^{3} of concrete (146 cm^2/m^3). No. 4 bars at 36-in. (914-mm) spacing are used as transverse reinforcement. Like most other states, Mississippi specifies the longitudinal reinforcement slightly above middepth. It is the intent of the Mississippi design that the specified value for longitudinal bars shall be 3¾ in. (95 mm) from center of the bar to top surface of the concrete. The unit of deviation shall be ±⅛ in. (3 mm). The lot size for conformance determination shall be 1000 ft (305 m) of pavement. Chair spacings shall not be greater than 36 in. (914 mm) c. to c. longitudinal and 27 in. (686 mm) transverse. Additional chairs shall be used if necessary to meet the steel placement requirements. The minimum length for laps is 20 in. (508 mm), and usually the laps are skewed (60 deg from center line).

At construction joints, longitudinal bars are required to extend a minimum length of 5 ft (1.74 m). No additional steel was installed at the construction joints on 5 projects built during 1962-1963. Since then No. 5 deformed bars, 5 ft (1.74 m) long and placed at 6½ in. (165 mm), have been used as additional steel.

For the first 5 years of practice, 5-lug anchors were used for bridge ends and 4-lug anchors for pavement ends. Since 1967, 4-lug anchors have been used for both bridge and pavement ends because the continuing measurements from the Jones County experimental project do not indicate any difference in pavement movement between the 4- and 5-lug anchor installations.

Current Practices in the Construction of CRCP

Base and subbase construction are the same for CRCP as for jointed pavement. The support of reinforcement on high chairs was the original installation method and has been an accepted standard for many years. In Mississippi, this method is the only one permitted with slip-form paving. Forms were used only on a few projects during the early 1960s.

Generally, materials, mixing, handling, and placing of concrete are no different for CRCP than for jointed pavement. Concrete usually has a cement factor of 1.40 (about 5.6 bags/yd^3 or 7 bags/m^3), which is the same as for jointed pavement, but only slip-form paving for CRCP uses air-entrained concrete. Air-entrained concrete contains no less than 3 percent and no more than 6 percent air. The limit for the concrete slump is between 1.5 and 2 in. (38 and 51 mm).

Proper vibration of the concrete is very important. The internal vibrations should be done in such a manner as not to dislocate the steel. The present Mississippi specification for consolidation of portland cement concrete pavements was written from the AASHO specification. For curing, both the white pigmented impervious membrane and white polyethylene sheeting have been used successfully. No more than 0.2 in. (5 mm) is allowed as tolerance for concrete thickness.

CURRENT DESIGN AND CONSTRUCTION PRACTICES

Splices

Splicing of reinforcing steel is a problem in CRCP. In the Mississippi design, minimum length for laps is 20 in. (508 mm) and usually the laps are skewed 60 deg from the center line. This design works fine for most projects. However, at the beginning of 2 projects where the adjacent project was completed several months earlier in another construction season, fragmental distress was found on top of the first lapped splices (Fig. 1). A few cores taken at random within the distressed area showed an excellent quality of concrete, but they were broken at the middle where the steel is located.

A close observation, made during the repair operation, indicated that the distress was not caused by construction and that the concrete was in good condition. Presence of the failure, which coincided with the lapped splices, suggested that it may be due, at least in part, to the very high tensile stresses in the steel ahead of the construction joint, which caused a decrease in bond at the splice where the concrete was poured several months later in another construction season. It is possible that the serious slab separation at the distressed area was preceded or accompanied by progressive reduction in bond between the closely spaced cracks.

It is recommended that, if a new project is to be added to an existing project that was constructed several months earlier in another construction season, the length of the first lapped splice should be more than the regular design of 20 in. (508 mm). The exact length of such lap is directly related to the local environment and is pending further study and experiments. An effort will be made to refer this special problem to the researchers at the University of Texas who are conducting a study on the design of CRCP. Before the new criteria become available, however, 30 in. (762 mm) is recommended for use at these locations. Consideration should also be given to adding extra cement to the first few batches of concrete at the beginning of the new project construction. This will allow mortar to coat the drums and truck and also give additional strength to the concrete immediately beyond the construction joint.

Transverse Reinforcement

Recent improvements in construction methods have decreased or eliminated the need to use transverse reinforcement to maintain spacing and depth of longitudinal steel (6). However, in Mississippi, the use of transverse reinforcement is still necessary. Experience has shown that the transverse reinforcement not only holds the longitudinal crack tightly closed but also keeps the cracked pavement from "sliding" due to the crown height of the pavement. This is especially true when expansive soils are present.

Terminal Treatments

The free end of a CRCP, regardless of its length, may undergo longitudinal movement and growth of as much as 2 in. (51 mm). Therefore, the free ends at the bridge and at the beginning or end of each individual project must have proper terminal treatments. Such terminals must be designed to restrain or to accommodate the movement.

In Mississippi, the only type of terminal treatment is the anchors for the purpose of restraining movement. Studies (20) have shown that these lug anchors will restrict approximately half the movement that would occur if no lugs were used. Therefore, the lugs are frequently used in conjunction with a few short reinforced concrete slabs connected by doweled expansion joints to absorb the additional movement. This lug-anchor system performed well in Mississippi.

The wide flange beam joint has been used successfully in many states (6). However, in Mississippi, no plan has been made to adopt this design because it is felt that the highly expansive subgrade soil in Mississippi may create other problems when the wide flange beam joint is used and thereby negate the original purpose of this type of design, i.e., to minimize maintenance costs and provide load transfer across the gap in the pavement under the flange.

End Construction Arrangement

Usually the plans specify that the pavement (except the splicing steel), base, subbase, and subgrade all end at the termination station of the project. For continuously reinforced concrete pavement, this created a problem for the construction of the adjacent project that would be constructed later. As shown in Figure 2, the contractor for the adjacent project has to bend the splicing steel in order to construct the base, subbase, and subgrade. Under this condition, the construction joint for the sublayers cannot be properly constructed to provide continuity and very often provides a weak plane when high stresses develop.

It is recommended that, when a continuously reinforced concrete pavement project is to be continued with another project, special end construction arrangement be made to allow continuity of all base structure layers. This can be accomplished by requiring the first contractor to extend the base layers 30 to 50 ft (9 to 15 m) beyond the termination station of the project.

Vibration of Concrete

Proper vibration of the concrete is very important, and both surface and internal vibration are used, sometimes in combination. The internal vibrations should be done in such a manner as not to dislocate the steel. Consideration should be given to adding extra cement to the first few batches of concrete each day. This will allow mortar to coat the drums and truck and also give additional strength to the concrete immediately beyond the construction joint. At the construction joint, additional hand vibrators should be used to ensure proper vibration (6).

A field test was conducted in Mississippi during the construction of a CRCP to study the effect of vibration on concrete (19). Research parameters included 3 vibrator frequencies (7,000, 9,000, and 11,000 impulses per minute in air), 1 eccentric weight (2.25 lb or 1.02 kg), and 5 paver speeds (10, 12, 14, 16, and 20 ft/min or 0.05, 0.06, 0.07, 0.08, and 0.10 m/s).

For each experimental section, 2 samples of fresh concrete, 1 behind and 1 between the vibrators, were taken for air content, unit weight, and gradation analysis. So that the lower half of the pavement (under the reinforcing steel) at the preselected location could be sampled, a portion of 2 longitudinal bars was omitted for a distance of 60 in. (1524 mm) (Fig. 3). This allowed the delineating sample device to take 4 increments of the sample above and 4 increments of the sample below the steel (Fig. 4). These samples were taken right behind the final finishing screed machine and rushed to the field laboratory for testing.

Random samples were taken from the aggregate stockpile (near the bin feeder belt) for gradation analysis.

A Rex slip-form paving machine was used on this project. There were 6 vibrators across a 12-ft (3.7-m) lane, and the diameter of eccentric in the vibrator was $1\frac{7}{8}$ in. (48 mm). Walking bridges were furnished by the contractor for use by technicians in sampling and testing. A FRAHM tachometer was used to measure and set the frequency of the vibrator (Fig. 5).

The fresh concrete was tested at the field laboratory for air content (AASHO T 152) and unit weight (AASHO T 121), and then the sample was washed over a No. 4 sieve and dried. Sieve analyses for the plus No. 4 aggregates were conducted at the central laboratory of the Testing Division. On-the-spot slump tests were conducted by the project office personnel. Nuclear density gauges were used at 4 different spots around the sample location to measure the density immediately behind the vibrator and between 2 vibrators. At each spot, readings were also taken at 2-in. (51-mm) depths and 6-in. (152-mm) depths to determine the density above and below the reinforcing steel (Fig. 6).

The nuclear gauge was prevented from sinking into the concrete and also protected from possible damage caused by the cement paste by a 2 × 2 ft (0.6 × 0.6 m) plywood board at each test spot. A $\frac{3}{4}$-in. hole was drilled in the plywood board to allow the nuclear gauge to take the direct transmission measurements at 2 in. (51 mm) and 6 in. (152 mm). The nuclear gauges, with the plywood board, were calibrated in the laboratory, and a special calibration curve was developed for this measurement.

On the finished pavement, cores were taken around the area of the sample location. These cores were completely submerged in water for 3 days, and impact tests were conducted. A model N Schmidt concrete test hammer was used for the impact tests.

Preliminary analysis indicated that the range of paver speeds studied had no noticeable effect on the densities. No significant effect of vibration on the strength of concrete was found. However, for the paving machine used on this project [$1\frac{7}{8}$-in. (48-mm) eccentrics at 24-in. (610-mm) spacing], vibration frequency of 9,000 impulses per minute (IPM) in air produced the highest density.

The degree of vibration had no significant effect on the entrained air content. The slump values above the 1-in. (25-mm) range (not to exceed 2 in. or 51 mm) allowed the vibrators to develop densities higher than would be developed were the slump below the 1-in. (25-mm) range.

The vibration effort studied on this project produced no evidence of segregation. The sieve analysis also indicated a very close conformity among the field samples (plastic concrete), stockpile samples, and original laboratory samples.

Figure 1. Fragmental distress at lapped splices on US-82 in Leflore County.

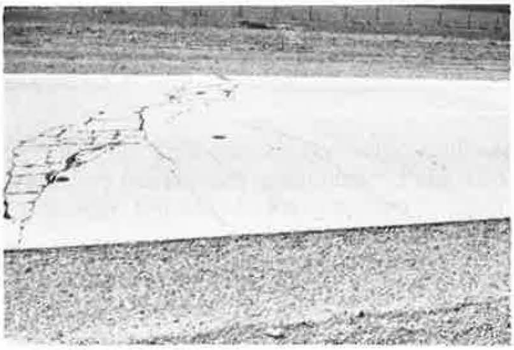


Figure 2. End arrangement of CRCP.

Figure 3. Location of samples for vibration study.

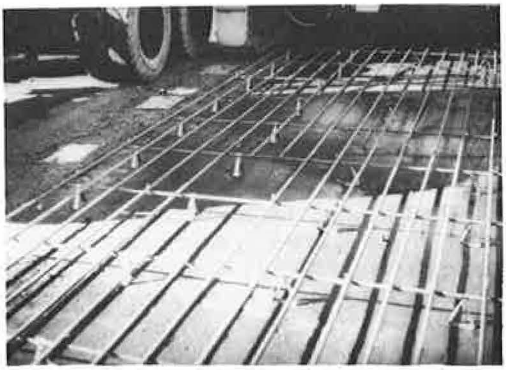


Figure 4. Field sampling for vibration study.

Figure 5. Using tachometer to set vibrator frequency.

Figure 6. Taking nuclear density measurements on plastic concrete.

The nuclear device is a very good tool for consolidation control for CRCP construction. Compared with the density obtained by the conventional method (AASHO T 121), the mean of the 4 nuclear density measurements was within ± 2.5 lb/ft^3 (± 40 kg/m^3) range accuracy. The standard deviation for all the nuclear density measurements was 1.13 lb/ft^3 (18 kg/m^3).

Curing of Concrete

White pigmented impervious membrane and white polyethylene sheeting have both been used successfully for the curing of concrete for continuously reinforced concrete pavement. Field surveys also indicated that these 2 curing methods did not appear to affect the pavement cracking pattern.

Longitudinal Center Joint

Before 1968, longitudinal sawed joints were used on all projects. Since that time, the polyethylene strip has been used for longitudinal center joints. When first adopted, the 4-mil (0.1-mm) strip was used. In 1969 the thickness of the strip was increased to 8 mils (0.2 mm). Some states have reported (3) that random longitudinal cracking has developed on projects having polyethylene strips at center joints. In Mississippi no such cracking has been found.

ANALYSIS OF FIELD MEASUREMENTS

Present Serviceability Index

Present serviceability index (PSI) was obtained on each of the 46 projects during the field surveys of 1970 and 1972. A PCA road meter was used to conduct the survey. The PSI and its relation to the accumulated equivalent annual 18-kip (8165-kg) single-axle loads is shown in Figure 7. As expected, the 18-kip (8015-kN) axle loads have significant influence on the PSI. In places where extremely high traffic counts or heavy traffic load is anticipated, it may be necessary to consider increasing the thickness of pavement.

The present equivalent annual 18-kip (80-kN) single-axle load applications (EALA) used in this report are not field measurements. They are calculated based on the following equation in the National Highway Functional Classification and Needs Study Manual of the Federal Highway Administration.

EALA = ADT × (percentage of total trucks and combinations) × (critical lane factor) × [18-kip (80-kN) single-axle equivalent constant] × 365

Another factor that is believed to have a very strong influence on the PSI is the differential movement of the very highly expansive subgrade soils. Pavements have been badly distorted and sometimes destroyed by the behavior of these active clay soils, which cover about 75 percent of the surface of the state. Seasonal wetting and drying have contributed to the roughness of the pavement surfaces through differential settlement and heaving of these active clay formations. Table 1 gives the Atterberg limits and specific gravities of major Mississippi clay formations.

Differential movement and distortion of pavement are often reported in the area of Delta Gumbo, Yazoo, and Zilpha Clay formations where the lowest PSI is recorded (Fig. 8). CRC pavements constructed on expansive soils have caused considerable maintenance problems. It is recommended that, unless specially designed, CRCP not be placed in areas of highly expansive soils.

Since only 2 measurements were made on each of the projects during the field survey, no attempt was made to make a detailed analysis of the annual change in the present serviceability indexes. However, on the average, the 1971 PSI readings were slightly lower than those of 1970.

Figure 7. PSI and accumulated equivalent annual 18-kip single-axle load (1 lb = 0.4536 kg).

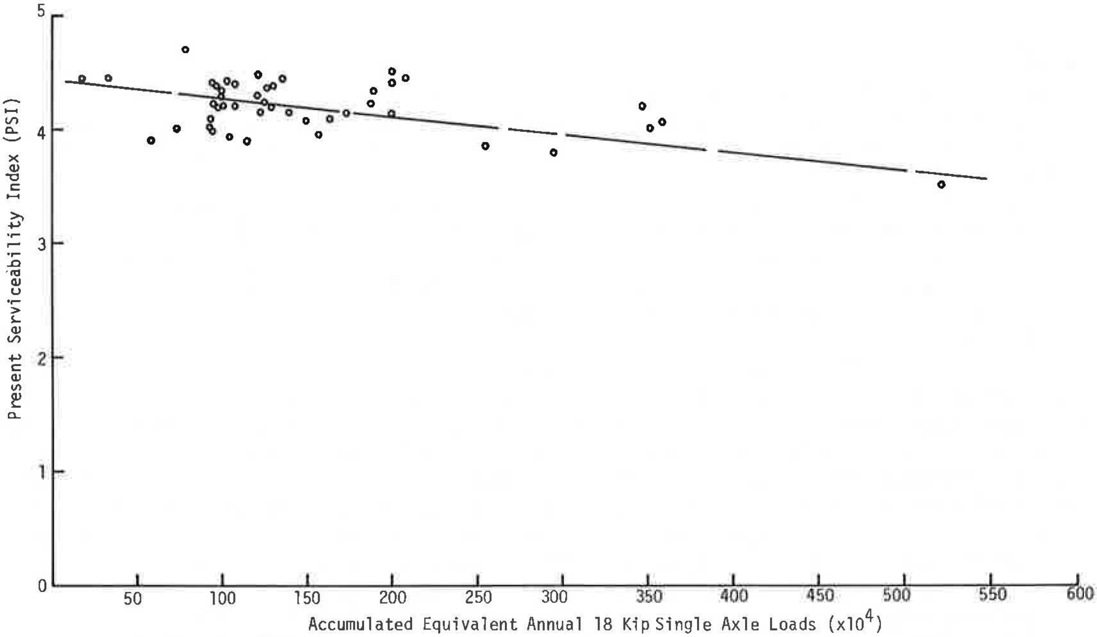
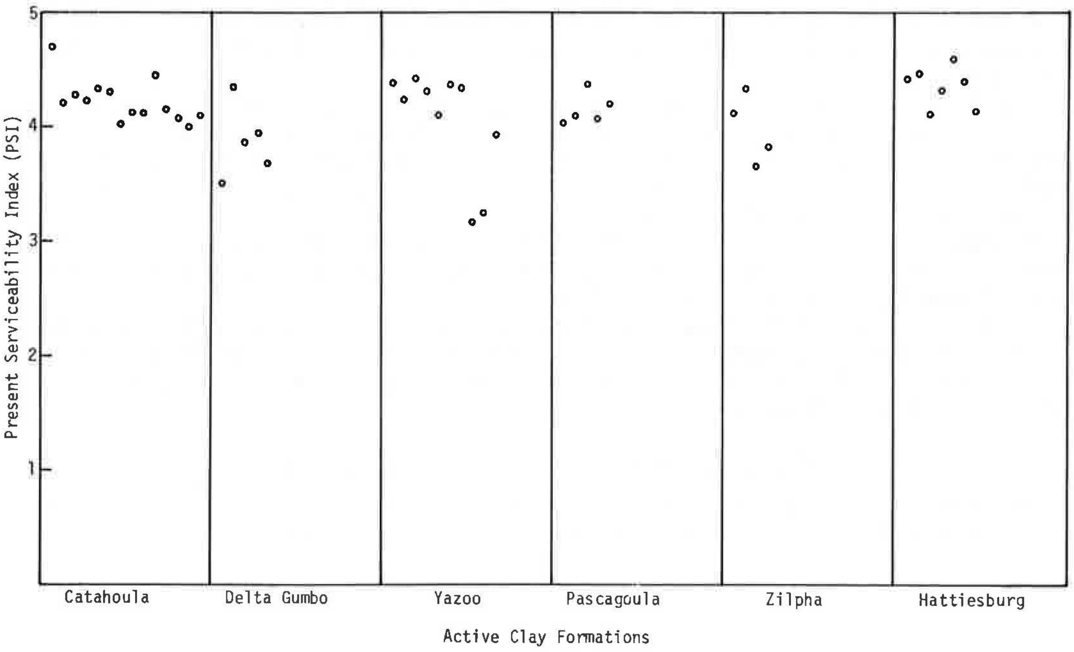


Table 1. Atterberg limits and specific gravity of major Mississippi clay formations.

Clay Formation	Liquid Limit	Plastic Limit	Plasticity Index	Shrinkage Limit	Specific Gravity
Catahouls	58	29	29	19	2.71
Delta Gumbo	94	31	63	20	2.77
Demopolis	56	27	29	19	2.77
Hattiesburg	40	18	22	17	2.72
Mooreville	52	24	28	16	2.73
Pascagoula	50	25	25	21	2.78
Porters Creek	86	44	42	18	2.74
Yazoo	112	33	79	24	2.82
Zilpha	100	42	58	30	2.75

Figure 8. Present serviceability index of Mississippi clay formations.



Cracking

By definition a continuously reinforced concrete pavement is a jointless pavement sufficiently reinforced with steel to develop a large number of transverse, hairline cracks that will not impair the structural integrity of the pavement but will reduce the maintenance costs.

The spacing of these hairline cracks has been found to be inversely proportional to the percentage of steel in the pavement. A spacing of about 3 to 10 ft (1 to 3 m) is desirable to produce acceptably small crack widths (6). Factors influencing the spacing and width of cracks are numerous (16). In this study an effort was made to determine whether any relation exists between crack spacing, pouring temperatures, and traffic data.

The crack survey is also a statewide effort. Ten percent of the total length from each project was picked at random for this purpose, but none of the random sections was near construction ends. Readings were taken from the edge of the roadway by visual observation. Any visible crack, large or small, was counted and recorded. Figures 9 and 10 show that the pouring temperatures and the traffic loads do not appear to dominate the crack spacing. Data in these 2 figures have a wide scatter, and any attempt to correlate them would be meaningless. Figure 11 shows present crack spacing versus the year of completion. The age of the pavement, after a few years of service, is not an important factor influencing the crack spacing. All projects, despite their year of completion, have a crack-spacing range from 2.5 to 4.5 ft (0.8 m to 1.4 m).

In Figures 9 through 11, crack spacing from Desoto and Jones Counties experimental projects were not used because of different methods used in counting the cracks. In the 2 experimental projects only the cracks extending all the way across the pavement were counted. On the other projects, any visible cracks, large or small, were counted.

During the course of the field survey, a project engineer conducted some informal research that indicated that the crack pattern of CRCP conformed very closely with the crack pattern of the cement-treated base. To verify this information, we selected two 400-ft (122-m) sections of roadway and made cracking patterns on the soil-cement-treated base and the CRCP. Figure 12 shows that the results are negative.

Deflection

Very limited information on the deflection of CRCP was obtained during the field survey by using the Dynaflect. The Dynaflect measures pavement deflection induced by an applied load. It is an electromechanical system consisting of a dynamic force generator, a motion measuring system that is mounted in a towed trailer, and 5 motion-sensing geophones suspended from the towing arm of the trailer. Dynaflect-measured deflections have good correlation with the Benkleman-beam measured deflections. It has been found that a Benkleman-beam deflection equals about 20 times the Dynaflect deflections (unit in mils).

Dynaflect deflection readings were conducted in 1972 on a few CRCP projects at an interval of every 0.1 mile (0.16 km). On the Desoto County experimental project that was built in 1971, the average deflection is 0.46 mil (0.012 mm); standard deviation = 0.098 mil or 0.0025 mm. In another CRCP project constructed in 1971, the average deflection is 0.45 mil (0.011 mm); standard deviation = 0.085 mil or 0.0022 mm.

On another selected CRCP project that was constructed in 1964 and that has since shown extra wide cracks and bad spalling, the average deflection is 0.75 mil (0.02 mm); standard deviation = 0.138 mil or 0.0035 mm.

Dynaflect readings were also obtained on 20 conventional portland cement concrete pavements that were constructed during the 1940s and 1950s. Average deflections are from 0.85 to 1.5 mil (0.022 to 0.038 mm); standard deviation ranged from 0.14 to 0.48 mil or (0.0036 to 0.012 mm).

MAINTENANCE OF CRCP—SLABJACKING

Generally, slabjacking is used for filling voids, raising the pavement where depressions occur, and stabilizing the distressed pavement area. However, this operation is

Figure 9. Crack spacing and average pouring temperature.

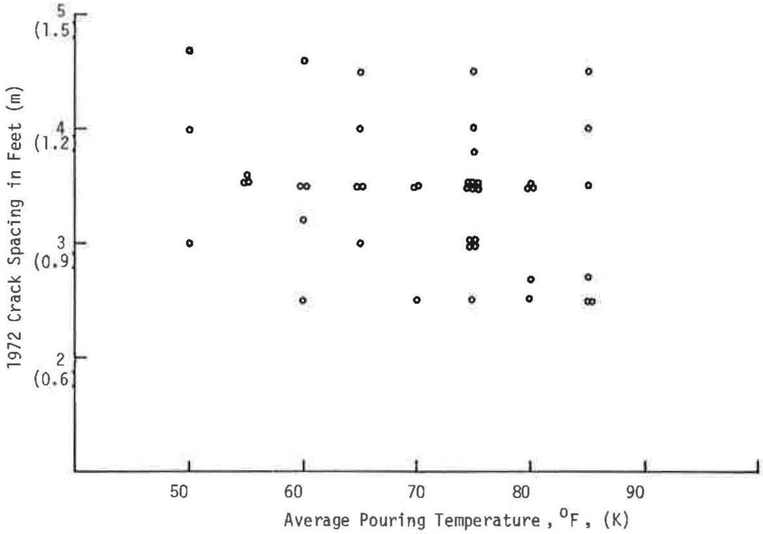


Figure 10. Crack spacing and accumulated equivalent annual 18-kip single-axle load (1 lb = 0.4536 kg).

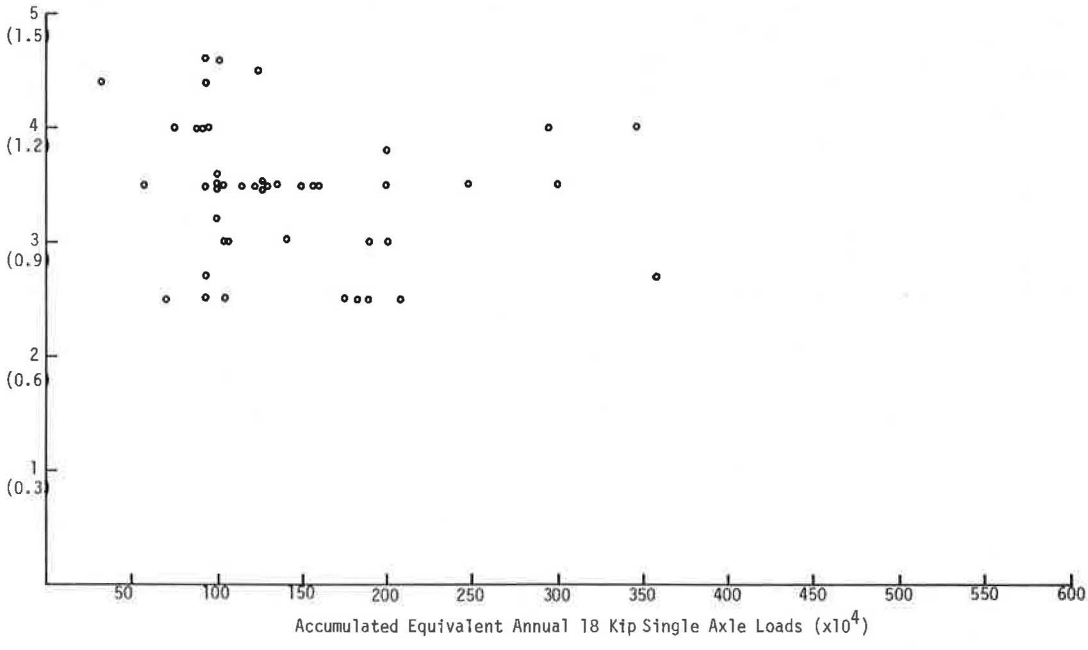


Figure 11. Crack spacing and year of project completion.

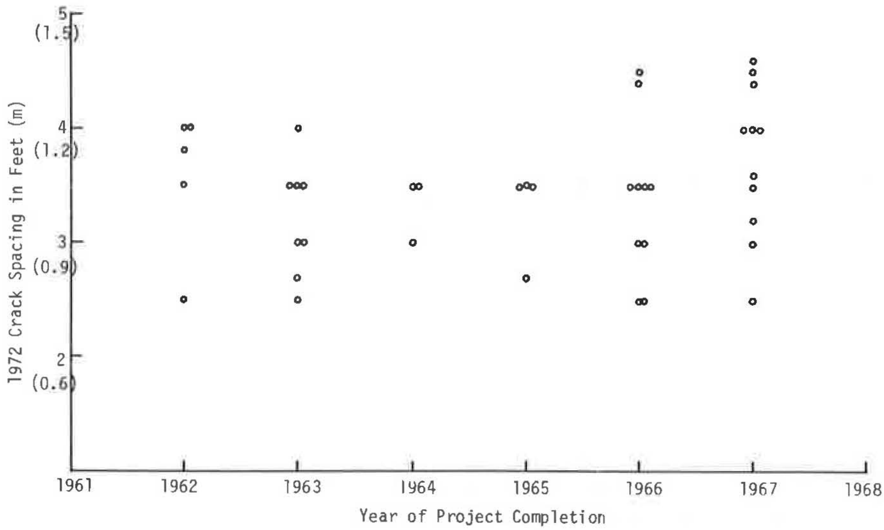
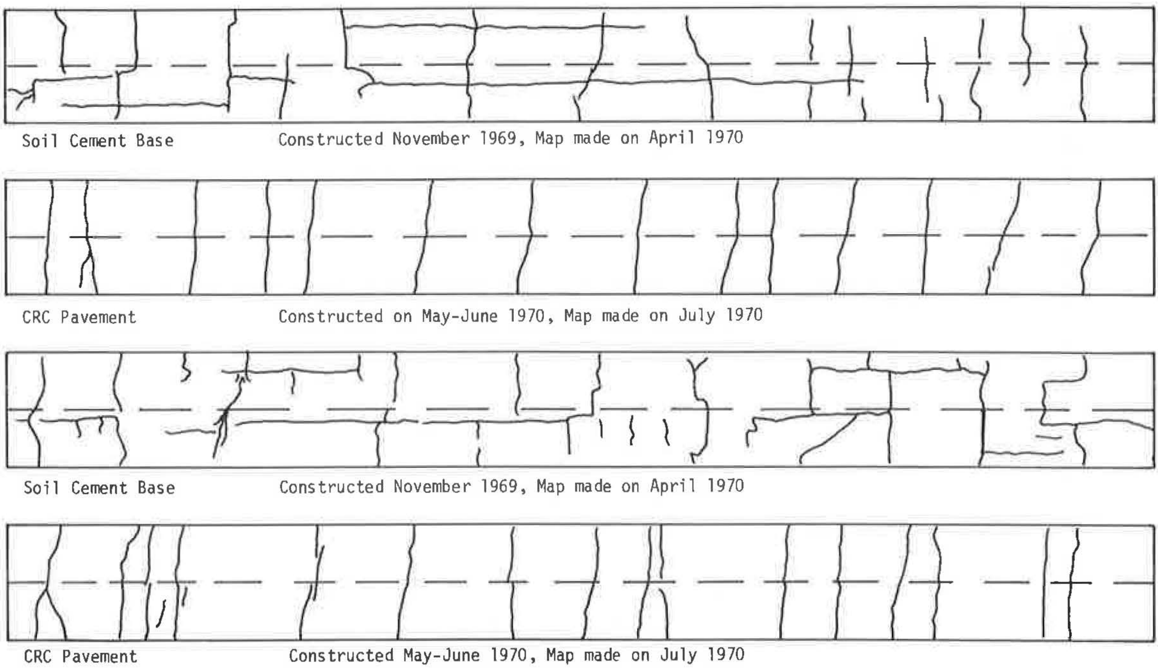


Figure 12. Cracking pattern of soil-cement base and superimposed continuously reinforced concrete pavement.



necessary for the maintenance of CRCP in Mississippi because of large areas of very active clay soils in the state. Seasonal wetting and drying have contributed to the roughness of the pavement surface through differential settlement and heaving of these active clay formations. The slabjacking operation can be used to feather out the grade on either side of the heaved or settled areas and thus maintain the riding quality of the pavement (22).

The slurry used in the slabjacking operation should be a mixture of high early strength portland cement (type 3), a minimum of 20 percent by volume; calcium chloride, a maximum of 5 percent by weight of cement; native silt soil or fine sand, a maximum of 75 percent by volume; and water. Type 1 portland cement can be used if type 3 is not available. However, when the type 1 is used, 28 percent cement should be added to the total mixture (13). Calcium chloride is to be added to the mixture in a premixed solution with water (21). Water should be added to produce the proper consistency.

On CRCP it is very important to drill the hole through base, subbase, and subgrade layer and to jack under the subgrade layer (4, 5, 9, 11, 14, 15, 22, 24). No more holes shall be drilled during a day's operations than can be jacked during the same day (5, 9). The best time for slabjacking is in cool weather (spring or fall) while the slabs are not at maximum expansion (14).

MAINTENANCE OF CRCP—TEMPORARY PATCH

Because of time required for the curing of portland cement concrete and interference with traffic, small areas of broken concrete resulting from CRCP distresses such as spalling, localized punch-out, construction joint, and localized radial are most often temporarily patched with bituminous mixture. When such patches are made, it is best to remove the loose concrete blocks before bituminous mixtures are applied. Usually this can be done by using a wrecking bar. Care should be exercised to avoid applying an excessive quantity of priming material; otherwise, rolling or shoving of the patch may occur (14).

On most locations, an initial temporary patch with bituminous mixture will provide good performance under traffic for a length of time. However, on some other locations this patch may need leveling or additional work annually or semiannually. It is a good practice to check these patches constantly to maintain the riding quality of the pavement.

MAINTENANCE OF CRCP—PERMANENT PATCH

For Small Distress Areas

On small distress areas where the failed material can be removed with a wrecking bar or other small tools, the fast-setting cement mix is a suitable solution for the permanent patch. The fast-setting cement can be mixed with pea gravel or used as a mortar. It also holds well when used as a skin patch. These patches will withstand traffic 30 to 40 min after completion.

For patches having firm bases or where the base is not damaged and the patches are from 4 to 8 in. (102 to 203 mm) deep, use 4 gal (0.015 m^3) of pea gravel per 50-lb (22.7-kg) bag of cement. For patches that are 2 to 4 in. (51 to 102 mm) deep, use 2 gals (0.0076 m^3) of pea gravel per bag of cement. For patches that are 1 to 2 in. deep (25 to 51 mm), just use the mortar and do not add any pea gravel. When this concrete mix is made, a water content not greater than 1.5 gal (0.0057 m^3) per bag of cement should be used. However, if the mixing temperature is higher than 80 F (300 K), this water content may have to be increased slightly to produce a workable mix. The initial water content has considerable effect on the 2-hour and the 7-day strengths but does not affect the long-term strength substantially. Therefore, the mix should always be placed and mixed with the least water possible to give a good working mixture (24).

This mixture hardens extremely fast, and it is very important that planning be done so that all the materials and equipment necessary to complete the patch are on hand when the first batch is mixed. This mixture will be hard enough to walk on in 15 min (24).

For Large Distress Areas

A standard maintenance procedure was established several years ago and has been used successfully by many districts and other highway departments. This procedure is applicable to large distress areas.

1. The existing pavement is sawed 1 in. (25 mm) deep normal to the centerline on both sides of the failure, and the failed concrete is broken out. The concrete must be removed for the full-lane width. To facilitate the removal of the broken concrete pavement, the reinforcing steel may be cut back to within 24 in. (610 mm) of the face of the remaining concrete.
2. The reinforcing steel replacing any bars that have been cut must have a minimum splice lap of 20 in. (508 mm), and the lapped steel is welded. The steel is not welded until the loose ends of the pavement in place have reached their maximum expanded movement. Usually this takes place around 3 to 4 o'clock in the afternoon.
3. Each end area of the patch is painted with epoxy resin.
4. High early strength cement should be used in the concrete placed in the patch. The concrete is normally placed as soon as possible after the reinforcing steel has been spliced. In no case should an area that has been opened for repairs be allowed to remain open overnight.
5. The concrete in the patch should be thoroughly cured.

When the outline of the patch is made, the edges of the proposed patched area (that will be sawed) should not cross or intersect existing cracks. Also, the saw cut should be no closer than 24 in. (610 mm) to the nearest crack.

When it is necessary to bend the steel at the 2 ends of the patch (to facilitate concrete removal), special precautions should be taken to make sure the steel is bent back to its original position. An S-shape resulting from such bending will sometimes create future problems.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

1. The age of the CRC pavement, after a few years of service, is not an important factor influencing the crack spacing. All projects, despite the year of completion, have crack spacing ranging from 2.5 to 4.5 ft (0.76 to 1.37 m).
2. In Mississippi, the pouring temperatures and the traffic loads do not appear to dominate the crack spacing.
3. Cracking pattern on the stabilized base does not influence the cracking pattern on the CRC pavement.
4. During paving operation, the paver speed does not have noticeable effect on the densities of concrete.
5. Vibration during construction had no significant effect on the strength of concrete.
6. The degree of vibration during construction had no significant effect on the entrained air content of the concrete.
7. Vibration frequency of 9000 IPM (in air) produced the highest density in concrete.
8. The slump values above the 1-in. (25-mm) range (not to exceed 2 in. or 51 mm) allowed the vibrators to develop densities higher than would be developed were the slump below the 1-in. (25-mm) range.
9. The vibration effort studied in this report (7000, 9000, and 11,000 IPM, in air) produced no evidence of segregation.
10. The sieve analysis indicated a very close conformity on the aggregate gradation between the plastic concrete, stockpile, and original laboratory samples.
11. The nuclear device is a very good tool for consolidation control of CRCP construction.
12. In Mississippi, white pigmented impervious membrane and white polyethylene sheeting have both been used successfully for the curing of CRCP. These 2 curing methods do not appear to affect the pavement cracking pattern.

13. Polyethylene strip center joint has been used successfully in Mississippi. No longitudinal cracking can be related to this type of center joint.

14. The 18-kip (80-kN) axle loads have a significant influence on the present serviceability index.

15. CRC pavements constructed on expansive soils have caused considerable maintenance problems. Seasonal wetting and drying have contributed to the roughness of the CRC pavement surface through differential settlement and heaving of these active clay formations.

16. CRC pavement on stabilized base and treated subgrade should have Dynaflect deflection measurements of about 0.45 mil (0.011 mm) (multiplied by 20 to get Benkleman-beam deflection measurements). For conventional portland cement concrete pavements, the average deflections are from 0.85 mil (0.022 mm) to 1.5 mils (0.038 mm).

17. Most CRCP in Mississippi is performing perfectly with tight cracks, visible only on close inspection and usually at normal spacing.

Recommendations

1. If a new CRCP project is to be added to an existing CRCP project that was constructed several months earlier in another construction season, the length of the first lapped splice should be more than the regular design of 20 in. (508 mm). Consideration should also be given to adding extra cement to the first few batches of concrete at the beginning of the new project construction.

2. When a CRC pavement project is to be continued with another project, special end construction arrangements should be made to allow continuity of all base structure layers. This can be accomplished by requiring the first contractor to extend the base layers 30 to 50 ft (9 to 15 m) beyond the termination station of the project.

3. The present specification on vibration of portland cement concrete should be revised, and the use of a tachometer to check or set vibrator frequencies should be adopted.

4. The use of a nuclear device (density gauge) for consolidation control should adopted.

5. CRC pavements, unless specially designed, should not be placed on areas with highly expansive clay soils.

6. The proposed CRCP maintenance procedure should be incorporated into the state's maintenance manual.

ACKNOWLEDGMENT

The authors wish to express appreciation to the many people whose efforts contributed to this report. Without this help it would not have been possible to accomplish the planned research. Grateful thanks are extended to personnel in all of the districts of the Mississippi State Highway Department, personnel in the Construction, Roadway Design, Testing, and Research and Development divisions of the highway department, and engineers of the Federal Highway Administration in the division, regional, and Washington offices. The opinions, findings, and conclusions expressed in this paper are those of the authors and not necessarily those of the Mississippi State Highway Department or the Federal Highway Administration.

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FINITE-ELEMENT ANALYSIS OF RIGID PAVEMENTS WITH PARTIAL SUBGRADE CONTACT

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A finite-element method programmed for a high-speed computer was developed for determining the stresses and deflections in concrete pavements with partial subgrade contact. The partial contact may result from the pumping and plastic deformation of the subgrade in combination with the upward warping of the slabs. The method is based on the classical theory of thin plates on Winkler foundations. The condition of contact is illustrated by spring analogies. If pumping and plastic deformation of the subgrade are negligible, the foundation is considered as a set of springs, the tops of which are all at the same elevation. If pumping and plastic deformation exist, the tops of the springs will be at different elevations, and that must be specified before an analysis can be made. The deformed shape of the slabs due to the combined effect of weight and warping is determined first and then used for computing the stresses and deflections due to wheel loads. The accuracy of the method for computing temperature stresses is verified by the assumption that the slab and the subgrade are in full contact and by making a comparison with Westergaard's exact solutions. The validity of the method in predicting the stresses and deflections in actual pavements is indicated by a comparison with the experimental measurements from the AASHO Road Test. The solutions based on partial contact check more closely with the experimental measurements than with those based on full contact.

•WHEN Westergaard (1) developed his theoretical method for computing the stresses in rigid pavements, he made 2 major assumptions. The first assumption is that the subgrade acts as a Winkler foundation consisting of a series of springs. The reactive pressure between the slab and the subgrade is directly proportional to the deflection with a constant of proportionality, which he called the modulus of subgrade reaction or k value. Although the k value is a fictitious quantity not characteristic of the subgrade only but depending also on other factors, the use of it greatly facilitates the analysis. Furthermore, the k value does not have a large effect on the stresses, and an approximate estimation will suffice for design purposes. The second assumption, which has been open to more skepticism, is that the slab and the subgrade are always in full contact. This assumption implies that the reactive pressure between the slab and the subgrade always exists no matter how the slab deflects. If the slab deflects upward at a certain point, the reactive pressure at that point will be downward, i.e., the subgrade will pull the slab down. This assumption is reasonable if each spring in the Winkler foundation functions perfectly and the weight of the slab imposes a uniform precompression on the spring. As long as the upward deflection is smaller than the precompression, the slab and the spring will be in contact. However, if some of the springs become defective because of pumping or plastic deformation, the assumption of full contact will no longer hold.

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Although Westergaard's analysis based on full contact can predict the stresses due to interior loading quite satisfactorily, it has great difficulty in predicting the stresses due to corner loading because the corner areas are normally not in contact with the subgrade. Results from the Arlington test (2) indicated that the pavement and the subgrade were not in full contact even if the slab was flat and there was no temperature differential between the top and the bottom. It was also found that the stresses in concrete pavements due to corner loading depended strongly on the condition of warping. When the corner was warped down and the slab and subgrade were in full contact, the observed corner stresses checked favorably with Westergaard's solutions. However, the observed stresses were 40 to 50 percent greater when the corner was warped up. Consequently, Westergaard's equation for corner loading was modified by Bradbury (3), Kelley (4), Spangler (5), and Pickett (6) to account for the loss of subgrade contact due to temperature warping, pumping, and plastic deformation of the subgrade. These modifications were based on empirical results, and no theoretical methods to the authors' knowledge have been developed so far. With the advent of high-speed computers and the finite-element method, it is now possible to analyze concrete pavements subjected to warping and loading without assuming that the slab and the subgrade are in full contact. It is believed that the method presented here is original and should provide a useful tool for the design and analysis of rigid pavements.

In a previous paper (7), the authors developed a finite-element method for analyzing concrete slabs with load transfer at the joints. The effect of partial contact between the slab and the subgrade was evaluated by simply deleting the reactive forces at those nodes that were assumed not to be in contact. In view of the fact that a node not in contact before loading might become in contact after loading and vice versa, it was indicated that, if the condition of contact at various nodes was specified, the stresses and deflections in the slabs could be determined by the finite-element analysis in which a method of successive approximations was used. It is the purpose of this paper to present such a method for analyzing concrete pavements with partial subgrade contact.

In this paper, the concept of full contact is discussed first. A general method, applicable to both full and partial contact, is then presented for computing the stresses and deflections in the slabs due to temperature warping. The accuracy of the method is verified by the assumption that the slab and the subgrade are in full contact and by a comparison with Westergaard's exact solutions. Next, 2 cases of partial contact are discussed. In the first case, there is no pumping or plastic deformation of the subgrade, so the springs in the Winkler foundation are all of the same length. Partial contact is caused by the upward warping of the slabs and occurs only near the pavement edges or corners. In the second case, there is pumping or plastic deformation of the subgrade, so the springs are assumed to be of unequal lengths. Partial contact is caused by the unequal length of the springs and possibly by the upward warping of the slabs. Finally, the finite-element solutions are compared with the experimental measurements from the AASHO Road Test. The solutions based on partial contact check more favorably with the experimental measurements than those based on full contact, thus indicating the validity of the method.

ANALYSIS BASED ON FULL CONTACT

The finite-element analysis of concrete pavements based on full contact and subjected to wheel loads was presented in the previous paper (7) and will not be repeated here. Only the case involving temperature warping is presented here. Before the theoretical formulation is presented, it may be worthwhile to discuss the concept of full contact by the spring analogy shown in Figure 1. Figure 1a shows a Winkler foundation consisting of a series of springs, each representing a nodal point in the finite-element analysis. When a slab is placed on the foundation, the weight of the slab will cause a precompression of the springs, as shown in Figure 1b. Because the slab is uniform in thickness, each spring will deform the same amount, and no stresses will be induced in the slab. The amount of precompression can be determined directly by dividing the weight of slab per unit area by the modulus of subgrade reaction. When the temperature is colder at the top of the slab than that at the bottom, as is usually the case at night, part of the slab will deflect upward, as shown in Figure 1c. However, the slab and the springs still remain in contact because the upward deflections are smaller than the precompression. The deflection of the slab due to warping can be determined by subtracting the precompression due to the weight of slab from the deflection due to the weight and the warping combined, as indicated by the shaded area in Figure 1c. The result is exactly the same as when the warping alone is considered. The same is true when a load is applied to a warped slab, as shown in Figure 1d. Therefore, when the slab and the subgrade are in full contact, the principle of superposition applies. The stresses and deflections due to warping and loading can be determined separately, one independent of the other, disregarding the weight of the slab. This principle forms the basis of Westergaard's analysis.

Stresses and Deflections Due to Warping

The general formulation involving warping is similar to that for loading, as described in the previous paper (7). After the stiffness matrix is superimposed over all elements and the nodal forces are replaced with the statistical equivalent of the externally applied loads, the following simultaneous equations can be obtained for solving the nodal displacements.

$$[K]\{\delta\} = \{F\} + k[A]\{\delta'\} \tag{1a}$$

where

$[K]$ = stiffness matrix of the slab,
$\{\delta\}$ = nodal displacements,
$\{F\}$ = nodal forces due to applied loads,
k = modulus of subgrade reaction,
$[A]$ = diagonal matrix representing the area over which subgrade reaction is distributed, and
$\{\delta'\}$ = subgrade displacements.

Note that the second term on the right side of Eq. 1a represents the nodal forces due to the subgrade reaction. If the slab has a total of n nodes, then

$$\{\delta\} = \begin{Bmatrix} \delta_1 \\ \vdots \\ \delta_i \\ \vdots \\ \delta_n \end{Bmatrix} \quad \{F\} = \begin{Bmatrix} F_1 \\ \vdots \\ F_i \\ \vdots \\ F_n \end{Bmatrix} \quad [A] = \begin{bmatrix} A_1 \cdots 0 \cdots 0 \\ \vdots \quad \vdots \quad \vdots \\ 0 \cdots A_i \cdots 0 \\ \vdots \quad \vdots \quad \vdots \\ 0 \cdots 0 \cdots A_n \end{bmatrix} \tag{1b}$$

and

$$\delta_i = \begin{Bmatrix} w_i \\ \theta_{xi} \\ \theta_{yi} \end{Bmatrix} \quad F_i = \begin{Bmatrix} F_{wi} \\ 0 \\ 0 \end{Bmatrix} \quad \delta_i' = \begin{Bmatrix} c_i - w_i \\ 0 \\ 0 \end{Bmatrix} \tag{1c}$$

where

i = subscript indicating the i th node;
w = vertical deflection, downward positive;
θ_x = rotation about x axis;
θ_y = rotation about y axis;
F_w = vertical force due to externally applied load, downward positive; and
c = initial curling of a weightless and unrestrained slab due to a temperature differential between the top and the bottom.

Note that $c = 0$ when there is no warping. The reason that F_i and δ'_i contain only one nonzero element is that the nodal forces are determined by statics and only vertical loads and reactions are involved.

Figure 2 shows, in an exaggerated scale, a thin slab subjected to a temperature differential, Δt, between the top and the bottom. If the slab is weightless and unrestrained, it will form a spherical surface with a radius R. Because the slab is only slightly curved, the length of the arc on the upper surface is practically the same as that on the lower surface, so the length L of the upper surface is shown as the length of the lower surface. The length is actually greater at the bottom than at the top by $\alpha L \Delta t$, where α is the coefficient of thermal expansion. Since the radius, R, is much greater than the thickness, h, and L much greater than $\alpha L \Delta t$, it can be easily shown from geometry that

$$R = \frac{h}{\alpha \Delta t} \tag{2}$$

and

$$c = \frac{d^2}{2R} \tag{3}$$

in which d = distance to the center of slab where curling is zero. Substituting Eq. 2 into Eq. 3 gives

$$c = \frac{\alpha \Delta t\, d^2}{2h} \tag{4}$$

Note that Δt is positive when the slab is warped up with a temperature at the top smaller than that at the bottom and negative when it is warped down.

The assumption that the slab remains in contact with the subgrade implies that the subgrade reaction always exists no matter how the slab is warped. If the slab is warped up, the subgrade will pull the slab down, and a deflection w is obtained, as shown in Figure 2. The displacement of the subgrade is thus c - w, as indicated by Eq. 1c. If w within $\{\delta'\}$ in the second term on the right side of Eq. 1a is moved to the left and combined with w on the left and c is combined with $\{F\}$, Eq. 1a becomes

$$[\overline{K}]\{\delta\} = \{\overline{F}\} \tag{5}$$

where

$[\overline{K}]$ = composite stiffness matrix of the system, and
$\{\overline{F}\}$ = composite nodal forces.

If there is no warping, then $c = 0$, and $\{\overline{F}\} = \{F\}$. After the displacements are obtained from Eq. 5, the stresses can also be computed.

The above derivation for upward warping also applies to downward warping. When the slab is warped down, the temperature differential is negative. If the temperature differential is the same for downward warping as for upward warping, the stresses and deflections will be the same in magnitude but opposite in sign.

Comparison With Westergaard's Solution

In the previous paper (7), the stresses and deflections due to wheel loads obtained by the finite-element method were checked with those by Westergaard's method and found to be in good agreement. To check the accuracy of the finite-element method for analyzing stresses and deflections due to temperature warping, a comparison with the Westergaard's solutions is also presented.

Westergaard (8) presented exact solutions for the stress and deflection due to warping in a concrete slab, $3\sqrt{2}\,l$ wide and infinitely long, where l is the radius of relative stiffness. His solutions for the stress and deflection along the y axis are shown by the 2 curves in Figure 3c. The stress and deflection are expressed as dimensionless ratios in terms of σ_0 and w_0 respectively, where

$$\sigma_0 = \frac{E \alpha \Delta t}{2(1 - u)} \tag{6a}$$

$$w_0 = \frac{(1 + \mu) \alpha \Delta t l^2}{h} \tag{6b}$$

where

E = Young's modulus of concrete, and
μ = Poisson's ratio of concrete.

In the finite-element analysis, a long slab, $3\sqrt{2}$ l wide and $12\sqrt{2}$ l long as shown in Figure 3a, is employed. Because of symmetry, only a quarter of the slab is used for the finite-element analysis. The slab is divided into rectangular finite elements, as shown in Figure 3b. The input data are

$E = 3 \times 10^6$ lb/in.^{2} (21 GN/m^2),
$k = 100$ lb/in.^{3} (27.2 MN/m^3),
$h = 9$ in. (230 mm),
$\alpha = 5 \times 10^{-6}$/deg F ($9 \times 10^{-6}$/deg K), and
$\Delta t = 10$ F (5.5 K).

The positive temperature differential indicates that the slab is warped up. The radius of relative stiffness, l, is 36.95 in. (938.5 mm), which is used for determining the actual size of grid for computation.

After the stress and deflection are computed, they are expressed as dimensionless ratios and shown in Figure 3c by the small circles. The stress is considered positive when the top of the slab is in compression, which is opposite to the sign convention employed by Westergaard. The finite-element solutions check closely with Westergaard's exact solutions, thus indicating that the finite-element analysis of temperature warping as developed by the authors is theoretically correct.

ANALYSIS BASED ON PARTIAL CONTACT

The major difference in procedure between full and partial contact is that it is not necessary to consider the weight of the slab in the case of full contact, but the weight of the slab must be considered in the case of partial contact. The latter case involves 2 steps. First, the gaps and precompressions of the subgrade due to the weight of the slab or due to the weight of the slab and the warping combined are determined. These gaps and precompressions are then used to determine the stresses and deflections due to applied loads.

It should be noted that full contact is a special case of partial contact. Every problem in partial contact is analyzed first by assuming that the slab and the subgrade are in full contact. If it turns out that they are actually in full contact, no iterations are needed. If some points are found out of contact, the reactive force at those points is set to zero. The process is repeated until the same contact condition is obtained.

Partial Contact Without Initial Gaps

This case applies to new pavements not subjected to a significant amount of traffic, such as the nontraffic loop in the AASHO Road Test. Each spring in the Winkler foundation is in good condition and, if the slab is removed, will rebound to the same elevation with no initial gaps, as shown in Figure 4a. Under the weight of the slab, each spring is subjected to a precompression, as shown in Figure 4b. If the slab is warped up, gaps will form at the exterior springs, as indicated by a positive s in Figure 4c, and precompressions will form at the interior springs, as indicated by a negative s. If the slab is warped down, all springs will be under precompression, as shown in Figure 4b, except that the precompressions are not equal. The displacements due to the weight of the slab and warping combined can be determined from Eq. 1, except that the

Figure 1. Spring analogy for full contact.

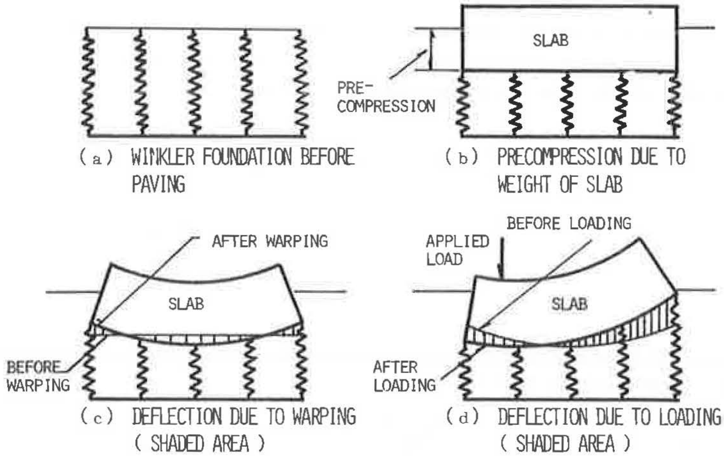


Figure 2. Warping of slab.

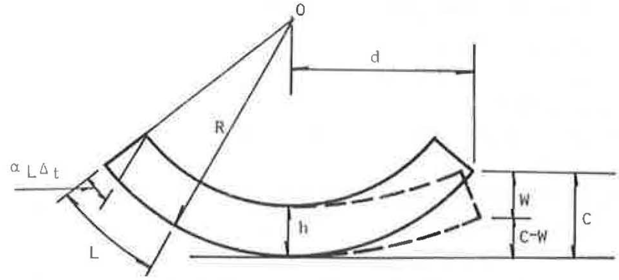
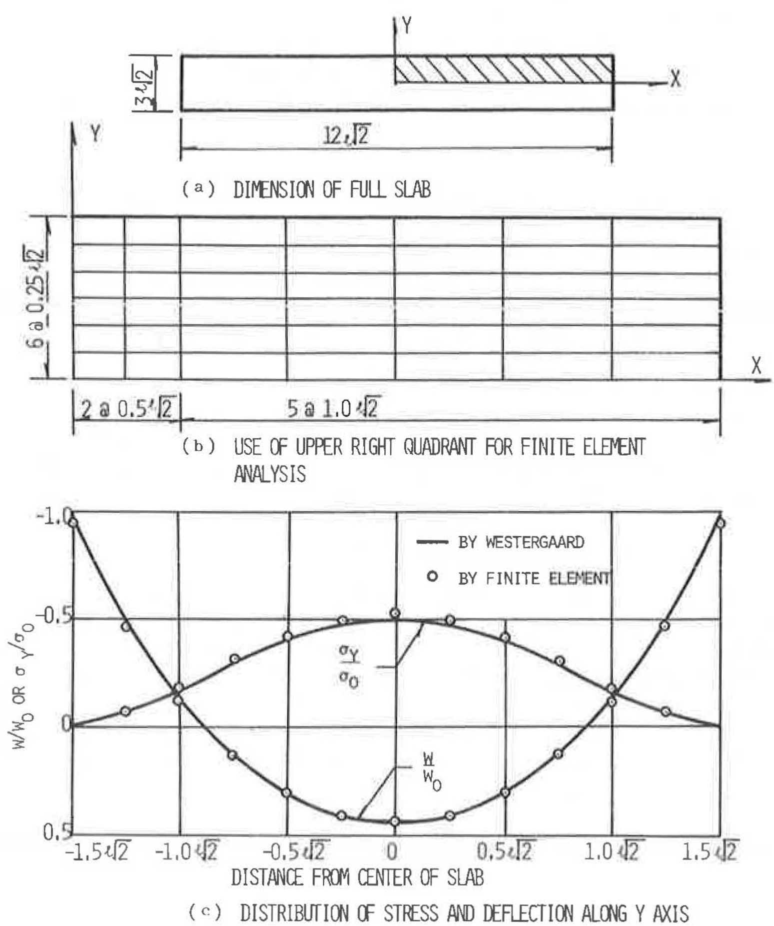


Figure 3. Comparison of finite-element solutions with Westergaard's exact solutions.



subgrade displacements are expressed as

$$\delta_i' = \begin{Bmatrix} c_i - w_i \\ 0 \\ 0 \end{Bmatrix} \qquad \text{when } w_i > c_i \tag{7a}$$

$$\delta_i' = \begin{Bmatrix} 0 \\ 0 \\ 0 \end{Bmatrix} \qquad \text{when } w_i < c_i \tag{7b}$$

Note that Eq. 7a is exactly the same as Eq. 1c for full contact and is used to start the iteration. After each iteration, a check is made on each nodal point to find whether any contact exists. If the deflection, w, is smaller than the initial curling, c, the slab is not in contact with the subgrade, and the subgrade displacement is set to zero, as indicated in Eq. 7b. Thus after each iteration, a new set of simultaneous equations is established. The process is repeated until the same equations are obtained. In most cases, this can be achieved by 5 or 6 iterations. After the deflections due to the weight and warping are determined, the gaps and precompressions can be computed and used later for computing the stresses and deflections due to the load alone.

To determine the stresses and deflections due to the load alone requires that the gaps and precompressions shown in Figures 4b or 4c, depending on whether warping exists, first be determined. When these gaps and precompressions are used as s, the deflections due to the load alone, as shown in Figure 4d, can be determined from Eq. 1, except that the subgrade displacements are expressed as

$$\delta_i' = \begin{Bmatrix} 0 \\ 0 \\ 0 \end{Bmatrix} \qquad \text{when } w_i < s_i \tag{8a}$$

$$\delta_i' = \begin{Bmatrix} s_i - w_i \\ 0 \\ 0 \end{Bmatrix} \qquad \text{when } w_i > s_i \text{ and } s_i > 0 \tag{8b}$$

$$\delta_i' = \begin{Bmatrix} -w_i \\ 0 \\ 0 \end{Bmatrix} \qquad \text{when } w_i > s_i \text{ and } s_i < 0 \tag{8c}$$

When w is checked with s, downward deflection is considered positive, upward deflection is considered negative, gap is considered positive, and precompression is considered negative. First, assume that the slab and the subgrade are in full contact and the deflections of the slab due to the applied load are determined. Then check the deflections with s and form a new set of equations based on Eq. 8. The process is repeated until the same equations are obtained.

When the slab and the subgrade are in partial contact, the principle of superposition no longer applies. To determine the stresses and deflections due to an applied load requires that the deformed shape of the slab immediately before the application of the load be computed first. Since the deformed shape depends strongly on the condition of warping, the stresses and deflections due to loading are affected appreciably by warping. This fact was borne out in both the Maryland (9) and the AASHO (10) road tests.

In the method presented here, the stresses and deflections due to weight and warping are computed separately from those due to loading. This is desirable because the modulus of subgrade reaction under the sustained action of weight and warping is much smaller than that under the transient load of traffic. If the same modulus of subgrade reaction is used, the stresses and deflections due to the combined effect of weight, warping, and loading can be computed in the same way as those due to weight and warping, except that additional nodal forces are needed to account for the applied loads.

Partial Contact With Initial Gaps

This case applies to pavements subjected to a high intensity of traffic, such as the traffic loops in the AASHO Road Test. Because of pumping or plastic deformation of the subgrade, some springs in the Winkler foundation become defective and, if the slab is removed, will not return to the original elevation. Thus, initial gaps are formed, as indicated by the 2 exterior springs shown in Figure 5a. These gaps, s, must be assumed before an analysis can be made.

The displacements due to the weight of slab, as shown in Figure 5b, can be determined from Eq. 1, except that the subgrade displacements must be expressed as

$$\delta_i' = \begin{Bmatrix} s_i - w_i \\ 0 \\ 0 \end{Bmatrix} \qquad \text{when } w_i > s_i \tag{9a}$$

$$\delta_i' = \begin{Bmatrix} 0 \\ 0 \\ 0 \end{Bmatrix} \qquad \text{when } w_i < s_i \tag{9b}$$

First, assume that the slab and the subgrade are in full contact. The vertical deflections of the slab are determined from Eq. 1. Then check the deflection at each node against the gap, s. If the deflection is smaller than the gap, as shown by the left spring in Figure 5b, Eq. 9b is used. If the deflection is greater than the gap, as shown by the other springs in Figure 5b, Eq. 9a is used. The process is repeated until the same equations are obtained. After the deflections are obtained, the gaps and precompressions can be computed and used later for computing the stresses and deflections due to loading, if no warping exists.

If the springs are of the same length, as shown in Figure 1, the weight of the slab will result in a uniform precompression, and no stresses will be set up in the slab. However, if the springs are of unequal lengths, the deflections will no longer be uniform, and stressing of the slab will occur.

Figure 5c shows the combined effect of weight and warping when the slab is warped down. The reason that downward warping is considered here, instead of the upward warping, is that the case of upward warping is similar to that shown in Figure 4c except that the gaps are measured from the top of the defective springs. Because the method is applicable to both upward and downward warping, the case of downward warping is used for illustration. The procedure for determining the deflections is similar to that involving the weight of slab alone except that the initial curling of the slab, as indicated by Eq. 4, is added to the gap shown in Figure 5a to form the total gap and precompression, s, for use in Eq. 9. Since the gap is either positive or zero and the initial curling may be positive or negative, depending on whether the slab is warped up or down, s may be positive or negative. After the deflections of the slab are obtained, the gaps and precompressions, as shown in Figure 5c, can be determined. These gaps and precompressions are used for computing the stresses and deflections due to the load alone, as shown in Figure 5d.

The above method was programmed for an IBM 360 computer, model 65, available at the University of Kentucky. The program can determine the stresses and deflections due to the weight of slab, the weight of slab plus warping, or the applied wheel loads in concrete pavements consisting of 1 slab, 2 slabs connected by a transverse joint, or 4 slabs connected by a longitudinal and a transverse joint. The efficiency of load transfer at each joint can be specified, and the same general principle, as described in the previous paper (7), is used in treating the doweled joints. To save the computer storage, load transfer is evaluated by an iterative procedure so that each slab is considered separately instead of all at the same time.

COMPARISON WITH AASHO ROAD TEST

In the previous paper (7), the edge stress under moving wheel loads and the stress distribution under vibratory loads, as measured in the AASHO Road Test, were compared with the finite-element solutions based on full subgrade contact. It was found that the finite-element method checked reasonably well with the experimental measurements if a k value of 300 lb/in.^{3} (81.5 MN/m^3) was used for moving loads and 900 lb/in.^{3} (244 MN/m^3) for vibratory loads. The agreement between the observed and the computed stresses based on full contact is not surprising because, unless the load is applied close to the corner of the slab, the condition of contact does not have a large effect on stresses. In view of the fact that the corner deflections are affected more significantly by the condition of contact, the corner deflections observed in the AASHO Road Test will also be presented and compared with the finite-element solutions.

Warping of Slabs Due to Temperature Differential

In the AASHO Road Test, the vertical movement of slabs due to the daily variation of temperature was measured on the nontraffic loop, and typical contours of vertical movement were published by the Highway Research Board (10). It is interesting to compare these measurements with finite-element solutions.

In the finite-element analysis for temperature warping, the following material properties are used: $E = 5.25 \times 10^6$ lb/in.^{2} (36.2 GN/m^2), $\mu = 0.28$, and $k = 50$ lb/in.^{3} (13.6 MN/m^3). The Young's modulus is the static modulus measured at the road test, which is smaller than the 6.25×10^6 lb/in.^{2} (43.1 GN/m^2) reported for the dynamic modulus. Although the k value obtained by the plate-bearing test at the road test varies from 63 to 135 lb/in.^{3} (17.1 to 36.7 MN/m^3), a smaller value is used here because the pressure applied to the subgrade due to temperature warping generally lasts for several hours instead of the 15 s used for the plate-bearing test.

In the AASHO Road Test, the temperature was measured in a 6.5-in. (165-mm) slab. The temperature at a point ¼ in. (6 mm) below the top surface of the 6.5-in. (165-mm) slab minus the temperature at a point ½ in. (13 mm) above the bottom surface was referred to as the standard temperature differential. The maximum standard temperature differential for the month of June and July, in which most of the warping measurements were made, averaged about -8.8 F (-4.9 K) when the slab is warped up and 18.5 F (10.3 K) when it is warped down. In line with the finite-element formulation, the sign convention for temperature differential used in this paper is opposite to that in the AASHO Road Test. A positive temperature differential always indicates that the temperature is smaller at the top than at the bottom. This convention applies hereafter whenever the standard temperature differential is referred to. Based on the finding of the Arlington test (2), there is a straight-line gradient in temperature between the upper and lower surfaces in the early morning and in the afternoon when the maximum temperature differentials occur. Therefore, the maximum temperature differential for a 6.5-in. (165-mm) slab at the road test should be 10 F (5.5 K) when the slab is warped up and -21 F (11.7 K) when it is warped down. Although temperatures in other thicknesses of slab were also measured in the AASHO Road Test, the data published by the Highway Research Board (10) are not detailed enough for use in an analysis. However, the published data do show that the temperature differential is not proportional to the thickness of the slab and that the increase in temperature differential is not so rapid as the increase in thickness. Based on the result of the Arlington test (2) that the maximum temperature differential of a 9-in. (230-mm) slab is 1.27 times greater than that of a 6-in. (150-mm) slab, the temperature differentials for 5-in. (130-mm), 9.5-in. (240-mm), and 12.5-in. (320-mm) slabs are calculated and given in Table 1. Only these 3 thicknesses are considered in this study because the stresses due to vibratory loads were measured on slabs of these thicknesses.

In the finite-element analysis of warping and loading, a 2-slab system rather than a 4-slab system is employed because a comparison between the 2 systems shows that the

Figure 4. Partial contact without initial gaps.

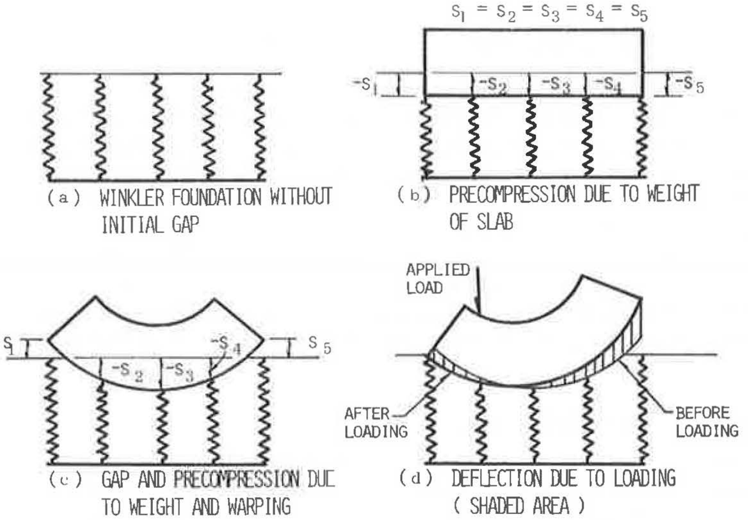


Figure 5. Partial contact with initial gaps.

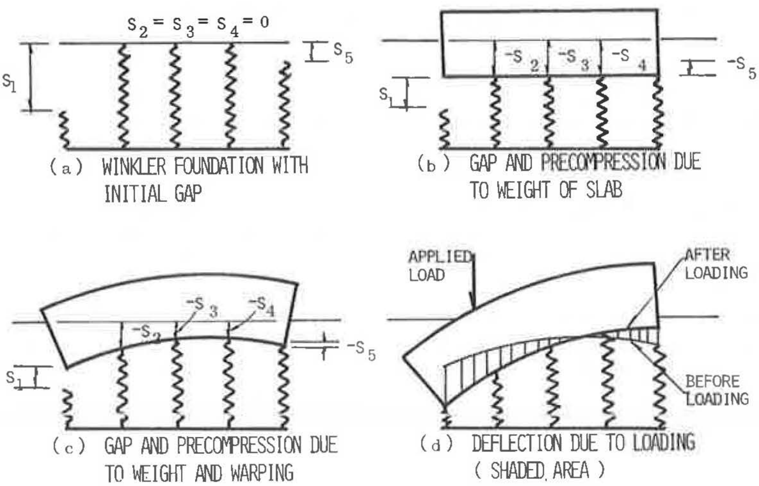


Table 1. Temperature differentials for analyzing AASHO Road Test rigid pavements.

Thickness (in.)	Warp Up (deg F)	Warp Down (deg F)
5	8.7	-18.3
9.5	12.7	-26.2
12.5	15.4	-31.4

Note: 1 in. = 25.4 mm; (deg F + 459.67)/1.8 = deg K.

2 slabs on the other side of the longitudinal joint have very little effect on the stresses and deflections at the corner region where measurements were made. The slabs are divided into finite elements with part of the nodal numbers and the 4 loading positions shown in Figure 6. Because of symmetry, only one slab is theoretically needed for the analysis of warping. However, 2 slabs are employed because the gaps and precompressions due to the weight of the slab and warping will be used for computing the stresses due to loading. As the experimental measurements were made on the nontraffic loop, the case of partial contact without initial gaps is assumed. The vertical movements are obtained in 3 steps. First, determine the gaps and precompressions due to the weight of the slab and the upward warping, as shown in Figure 4c. Then, determine the gaps and precompressions due to the weight of the slab and the downward warping. Finally, obtain the vertical movements by subtracting the former from the latter.

Figure 7 gives a comparison of the vertical movements measured at the road test with those computed by the finite-element method. The solid lines are the finite-element solutions based on full contact, the crosses are those based on partial contact, and the small circles are the experimental measurements obtained from the typical contours published by the Highway Research Board (10). Starting from left to right, the figure shows the vertical movements along the pavement edge, the diagonal of a corner, and the transverse joint. The numerals along the abscissa refer to the nodal number shown in Figure 6.

Under the temperature differentials assumed, most parts of the slabs, except that at the extreme corner, remain in contact with the subgrade, so there is very little difference between full and partial contact. The theoretical movements are somewhat smaller than the observed movements, but the general trends are similar. In view of the fact that the assumed temperature differentials may not be the same as those when measurements were made and that the actual k value may be different from the 50 lb/in.^{3} (13.6 MN/m^3) assumed, the agreement between the theoretical solutions and the experimental measurements should be considered satisfactory.

The results of the 9.5-in. (240-mm) slab is left out purposely because the experimental measurements were inconsistent. The experimental data show that the movements of the 9.5-in. (240-mm) slab are generally greater than those of the 5-in. (130-mm) and 12.5-in. (320-mm) slabs in the 15-ft (4.6-m) nonreinforced sections but smaller in the 40-ft (12.2-m) reinforced sections. This fact was pointed out in the AASHO report (10), but no explanation could be made. The theoretical solutions based on a 15-ft (4.6-m) panel check well with the experimental measurements on the 40-ft (12.2-m) panel, but not so good with those on the 15-ft (4.6-m) panel.

Stress Distribution Under Vibratory Loads

In the previous paper (7), a comparison of stresses under vibratory loads was made between the finite-element solutions based on full contact and the experimental measurements. It was found that, when E = 6.25×10^6 lb/in.^{2} (43.1 GN/m^2), $\mu = 0.28$, and k = 900 lb/in.^{3} (244 MN/m^3), the finite-element solutions checked quite well with the experimental measurements, except for the minor principal stress in the 5-in. (130-mm) and 9.5-in. (240-mm) slabs. It was indicated that these discrepancies were due to the warping of slabs because the experimental data were taken during the early morning hours when the corners and edges of the slab were warped upward and part of the slab was not in contact with the subgrade. With the capability of evaluating the effect of warping on loading, as developed in this study, it will be interesting to find the effect of upward warping on the theoretical stresses. Because the stresses were measured on the nontraffic loop, the case of partial contact without initial gaps is assumed. The gaps and precompressions due to the weight of slab and the upward warping, as shown in Figure 4c and obtained in the previous section, are used for computing the stresses due to the applied load.

Figure 8 shows a comparison of the theoretical and experimental distribution of major and minor principal stresses. The figure is the same as that presented in the previous paper (7), except that the results based on partial contact are included. Four loading

Figure 6. Loading positions and finite-element grid.

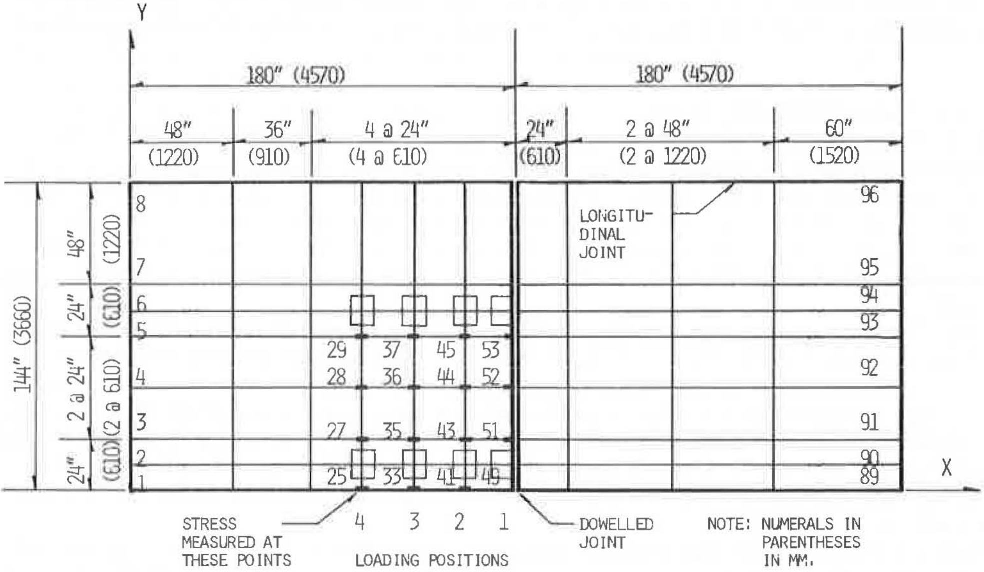
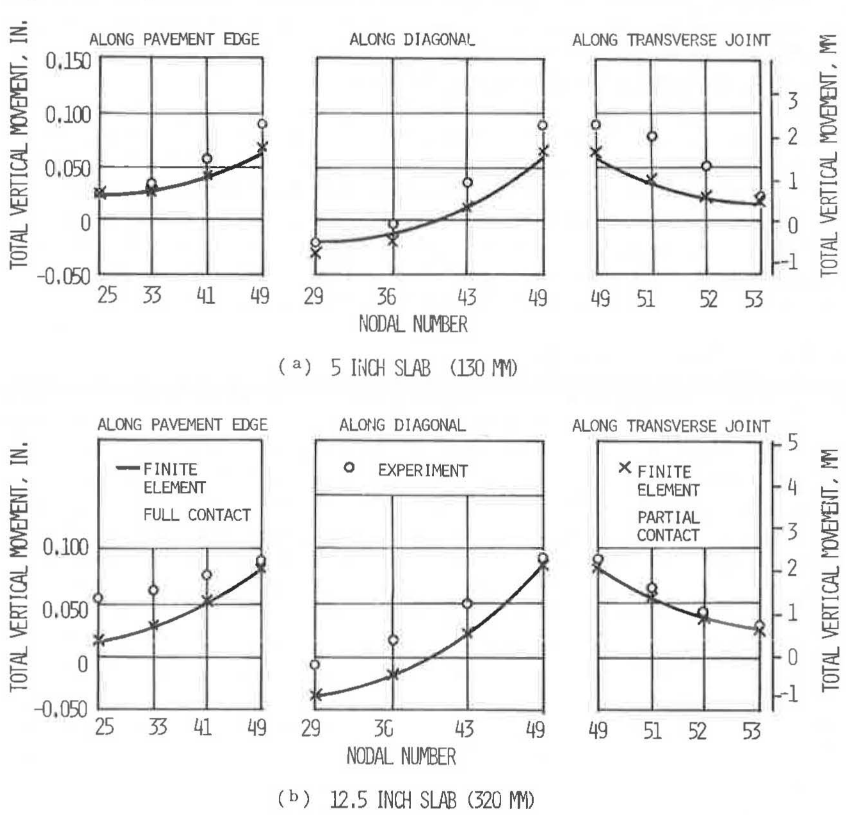


Figure 7. Comparison of theoretical and observed vertical movements due to warping.



positions, as shown in Figure 6, are considered. For loading position 1, the stresses are those along the joint. For other loading positions, they are along a line connecting the centers of the 2 pads. The figure shows that a better agreement between the theoretical solutions and the experimental measurements is obtained if the effect of warping is considered. The significant improvement in the minor principal stresses for the 5-in. (130-mm) slab is attributed to the fact that warping causes a loss of contact over a large area near the corner of the slabs. The effect of warping is quite small for the 12.5-in. (320-mm) slab because only a small area near the corner is not in contact. The figure also shows that upward warping has a large effect on the minor principal stresses when the load is applied near the corner, as indicated by loading position 1, but very little effect on both the major and minor principal stresses when the load is far from the corner.

Corner Deflections in Pavements on Traffic Loops

All previous comparisons between the finite-element solutions and the experimental measurements are made on the nontraffic loop, and the solutions are based on partial contact withoug initial gaps. To demonstrate the applicability of the method to actual pavements with initial gaps, we compute by the finite-element method the corner deflections at the traffic loops due to an 18-kip (80-kN) single-axle load and compared them with the rebound deflections measured by a Benkelman beam. The following material properties were used in the analysis: $E = 6.25 \times 10^6$ lb/in.^{2} (43.1 GN/m^2), μ = 0.28, and k = 300 lb/in.^{3} (81.5 MN/m^3). These values are the same as those used in the previous paper (7) for analyzing the dynamic edge stress and are based on the transient nature of the rebound measurements.

The 2-slab system is divided into finite elements, as shown in Figure 9. The tire imprints are converted into 4 rectangular areas, each 9.93 in. (252.2 mm) by 6.83 in. (173.5 mm). The spacing between is 11.5 in. (292.1 mm). The thickness of slabs is assumed to be 6.5 in. (165 mm). Three temperature differentials, i.e., 10, 0, and -21 F (5.5, 0, and -11.7K), are employed, which correspond to standard temperature differentials of 8.8, 0, and -18.5 F (4.9, 0, and -10.3 K) respectively. The standard temperature differential is the difference in temperature between 2 points in a 6.5-in. (165-mm) slab, one ¼ in. (6 mm) below the top and the other ½ in. (13 mm) above the bottom.

Figure 10 shows a comparison of the finite-element solutions and the experimental measurements. The smooth curve, which represents the relation between corner deflections and standard temperature differentials, is obtained from an empirical equation developed from the AASHO Road Test (10) for a slab of 6.5 in. (165 mm). In both the case without initial gaps and the case with initial gaps, the gaps and precompressions due to the weight of the slab and the warping, if the latter exists, are determined first and then used for computing the stresses and deflections due to the axle load. The average computer time is 40 s for analyzing the weight and warping for each temperature differential and 35 s for analyzing the axle load.

Figure 10 shows that the finite-element solutions based on partial contact without initial gaps do not check with the experimental measurements. If no initial gaps exist between the slab and the subgrade, the corner deflections due to the applied load are nearly the same no matter whether the slab is flat with a 0 temperature differential or is warped down with a negative temperature differential because full contact prevails in both cases. However, if gaps are assumed to exist along the edges and joints, the results, as indicated by the crosses, are much improved and check closely with the experimental measurements. The fact that the gaps have practically no effect on corner deflections at a standard temperature differential of -18.5 F (-10.3 K) is that at this temperature differential the gaps are not large enough to cause a loss of subgrade contact at the corner. If larger gaps are provided so that the corner of the slab is not in contact with the subgrade, the corner deflection will certainly increase.

The initial gaps assumed in the analysis are 0.04 in. (1mm) along the edge and 0.02 in. (0.5 mm) along the joint. These gaps are the results of pumping and plastic deformation of the subgrade and should be greater at the edge than at the joint because under a

Figure 8. Comparison of theoretical and experimental distribution of major and minor principal stresses.

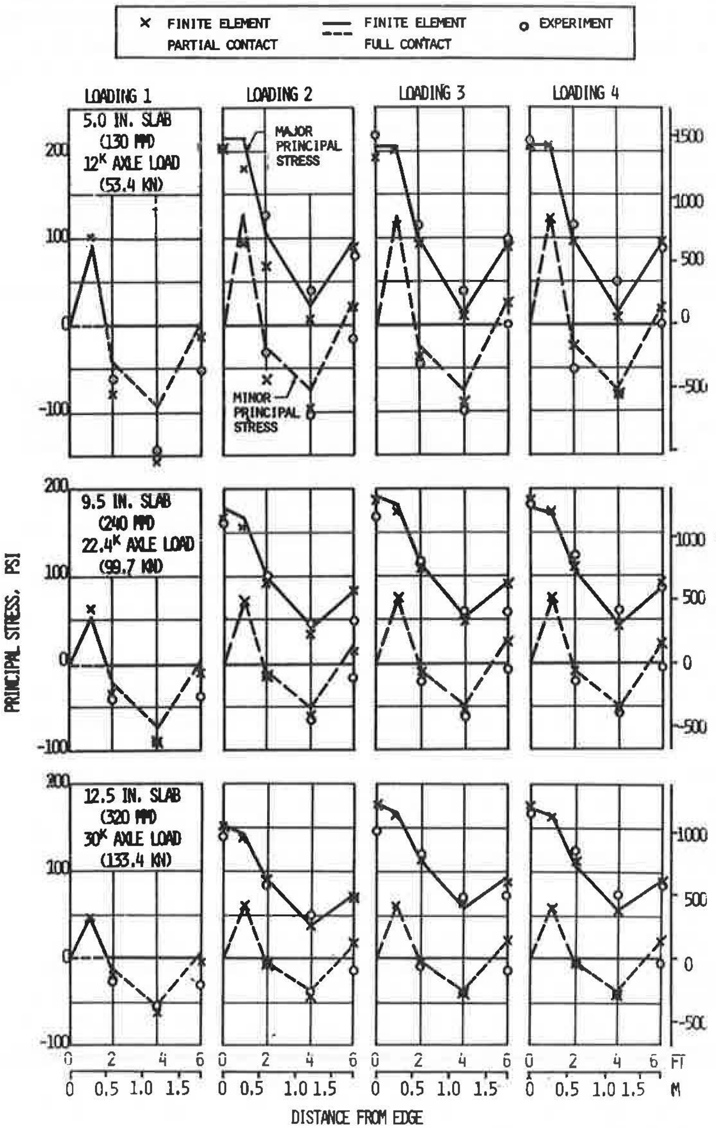


Figure 9. Finite-element network and location of 18-kip single-axle load.

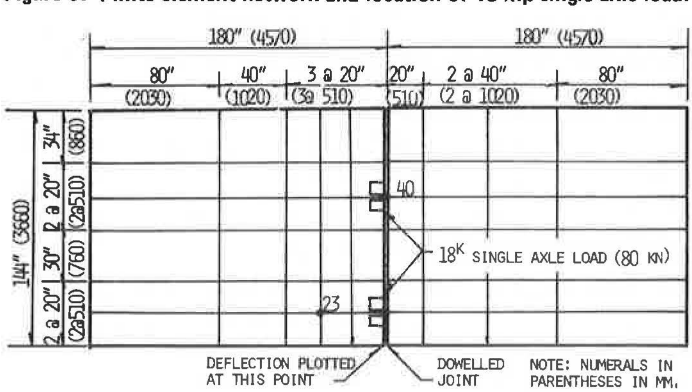
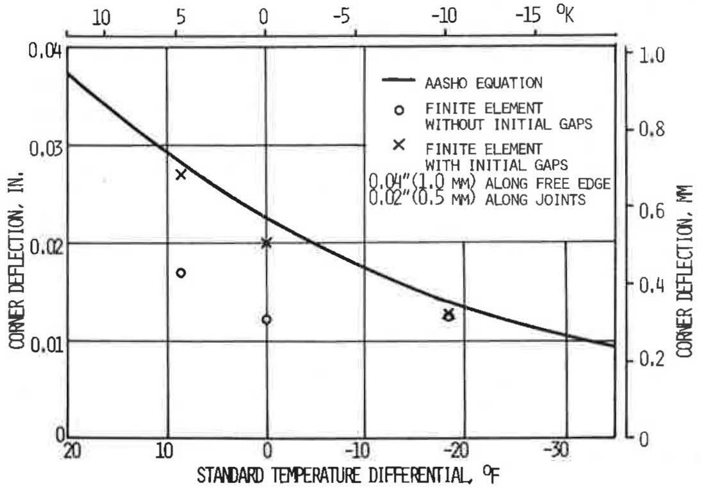


Figure 10. Corner deflections due to 18-kip single-axle load.



given wheel load the deflection is greater at the edge than that at the joint. Although the use of the above gaps yields results comparable to the experimental measurements, it is possible that for other pavements, in addition to the edge and joint, initial gaps also exist at other points near the edge and joint. The amount and distribution of these initial gaps depend on a variety of factors, such as type of subgrade, magnitude and repetitions of wheel loads, thickness of slab, and climatic conditions. Further research is needed to ascertain the amount and distribution of these gaps so that they can be used directly for determining the stresses in concrete pavements.

SUMMARY AND CONCLUSIONS

A finite-element method programmed for a high-speed computer was developed for analyzing concrete pavements with partial subgrade contact. The partial contact may result from the pumping and plastic deformation of the subgrade in combination with the upward warping of the slab. The lack of a theoretical method for analyzing this practical problem has plagued highway engineers for several decades. It is believed that the method presented here provides an efficient tool for solving this complex problem.

The method presented is based on the classical theory of thin plates on Winkler foundation. The accuracy of the method for computing temperature stresses is verified by assuming that the slab and the subgrade are in full contact and by making comparisons with Westergaard's exact solutions. The validity of the method in predicting the stresses and deflections in actual pavements is indicated by comparisons with the experimental measurements from the AASHO Road Test. The method has potential application in the design of rigid pavements.

Based on the condition of contact, the analysis of rigid pavements is divided into 3 cases: full contact, partial contact without initial gaps, and partial contact with initial gaps. In the case of full contact, the stresses and deflections due to wheel loads can be computed directly, disregarding the weight and warping of slabs; in the 2 cases of partial contact, the deformed shape of the slabs due to the combined effect of weight and warping must first be determined and then used for computing the stresses and deflections due to wheel loads. A comparison between the finite-element solutions and the experimental measurements made on the nontraffic loop at the AASHO Road Test clearly indicates that, if no initial gaps exist between the slab and the subgrade, the assumption of full contact generally yields reasonable results, even if the slab is warped up. However, in some cases the agreement is improved when partial contact is assumed. If there are initial gaps between the slab and the subgrade as a result of pumping and plastic deformation, the assumption of full contact is no longer valid, and the method based on partial contact with initial gaps should be employed.

A major contribution of this paper is the development of a method that takes into consideration the effect of warping on the stresses and deflections due to wheel loads. The application of the method based on partial contact without initial gaps is straightforward. Compared with the method based on full contact, it requires very little additional information other than the temperature differential between the top and bottom of the slabs. However, the applications of the method based on partial contact with initial gaps is more difficult because it requires a prior knowledge of the amount of gaps at various points beneath the slabs. These gaps are larger near the pavement edges and joints and become negligible in the interior of slabs. They must be properly estimated before an analysis can be made.

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SENSITIVITY ANALYSIS OF THREE RIGID PAVEMENT DESIGN TECHNIQUES

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Three rigid pavement design techniques were studied to determine the relative effects of various design parameters on pavement thickness. The analysis phase of this investigation consists of formulating each technique into a comprehensive mathematical or graphical thickness model. An evaluation of the influences on thickness of the major design factors was accomplished by a sensitivity analysis with 1 theoretical and 2 practical measures of parameter importance. The theoretical measure reveals considerable differences among techniques as to the process of resolving design thicknesses and as to the relative theoretical influence of various parameters that estimate the same major design factors. Practical measures of parameter importance, which account for variations in parameter values as well as the manner in which the design variables are formulated in the thickness model, show a greater consistency in the importance of generic factors among design methods. Although the relative importance of the traffic load differs with the 3 design methods investigated, this parameter and the flexural strength of the pavement are the influential factors in the design of rigid pavement thicknesses.

•THE OBJECTIVE of rigid pavement design is the provision of an acceptable riding surface that can withstand the deteriorating effects of traffic and environment for the service life of the facility. This goal is considered an integral part of the total highway transportation program and is constantly sought in quantitative measures. Various measurable parameters are used to quantify the physical demands imposed on the pavement structure and the subjective desires of road users for a good riding surface. As a result, several design techniques have been developed to combine in a logical manner these design parameters to determine the required pavement thickness.

Pavements designed for the same traffic load, soil support, and environmental conditions, built of the same quality of materials and workmanship, and expected to exhibit similar performance characteristics should be equal in thickness regardless of the agency responsible for the design and construction. Such agreement is not the case because, in addition to the differences of the various design procedures available, much engineering experience and subjective judgment enter into the decisions required for resolving the design of pavement thicknesses.

Determining a satisfactory design thickness is inherently difficult, and the optimum design cannot be ascertained even though the major factors affecting pavement thickness have been identified. Part of this problem is due to the uniqueness of the road structure and the conditions under which the facility must serve. A pavement is a thin narrow structure relative to its length and is built at or near the ground surface. Thus, a single design, which is seldom changed during the contracted length of the roadway, must satisfy a variety of subgrades and environmental influences. The heterogeneous nature of pavement-building materials and their changing behavior with time and ambient conditions also contribute to the uncertainty of the pavement design

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process. Although design is logically influenced by the performance expected of the pavement, methods of pavement evaluation and definitions of failure conditions have not always been clearly established for economical engineering purposes.

The purpose of this research was to identify and examine the relative effects on design thickness of various design parameters that measure subgrade support, traffic load, pavement material properties, environmental factors, and performance criteria (2). Three rigid pavement design techniques, which are representative of present design practices, were analyzed to define the important set of pavement design variables considered in each design method. The relative importance of these design factors was determined in a sensitivity analysis that was developed to investigate the impact of changes in parameter values on the rigid pavement structure.

PROCEDURE

The procedure used in the analysis of selected rigid pavement design techniques is subdivided into 3 phases: selection of pavement design methods, modeling of the design techniques, and sensitivity analysis.

Selection of Pavement Design Methods

Because numerous techniques exist for the design of rigid pavements, selection of several design methods was necessary to carry out this research investigation. Common usage of the technique, a rational approach to resolving the design thickness, and the availability of literature pertaining to the design method were the main criteria used in selecting the 3 design methods for analysis (5). The rigid pavement design methods that best satisfy these criteria and that are representative of present design practices are those issued by AASHO (1), the Corps of Engineers (4), and the Portland Cement Association (3).

Modeling of the Design Techniques

After a thorough review of the literature pertaining to the selected pavement design methods, models of each design method were formulated to mathematically or graphically systematize the design-variable relations for the purpose of executing the sensitivity analysis. Design charts to facilitate the determination of the required pavement thickness and mathematical equations that are basic to the various design processes are available for each method chosen for this research investigation.

Sensitivity Analysis

The effects on thickness of the factors considered in pavement design were quantitatively evaluated in a sensitivity analysis. Applied to each design technique investigated, the sensitivity analysis basically examined the change in design thickness produced by changes in the various design parameters.

The underlying premise of the sensitivity analysis is that, as the change in required pavement thickness produced by a design parameter becomes larger, the more important that parameter becomes in the design method. On this basis, the rate of change of thickness with respect to a particular parameter (quantified by the first partial derivative of the thickness with respect to that parameter) defines the theoretical measure of parameter importance. On the other hand, the magnitude of the thickness change produced by comparable parameter variations specifies the practical measure of parameter importance.

Theoretical Measure of Design Parameter Importance—The relative theoretical importance is defined as the positive ratio of the partial derivative of a thickness with respect to a particular parameter to the sum of the absolute values of the partial derivatives for all parameters. This determination is symbolically written as

$$RPP = 100 \times \left| \frac{\partial T}{\partial P_i} \right| \div \sum_{i=1}^{n} \left| \frac{\partial T}{\partial P_i} \right| \tag{1}$$

where

RPP = relative percentage of a partial,

$\left| \frac{\partial T}{\partial P_i} \right|$ = absolute value of the first partial derivative of the thickness function T with respect to the design parameter P_i, and

$\sum_{i=1}^{n} \left| \frac{\partial T}{\partial P_i} \right|$ = summation of the absolute values of all partial derivatives considered in the sensitivity analysis of a design method.

The above relative percentage adequately determined the importance of each parameter as formulated in the thickness function, but this technique implicitly assumes that parameter variations are numerically "small" and equal.

Measures of Practical Importance of Design Parameters—The sizes of parameter variations encountered in actual pavement design practice differ considerably among design parameters, and these increments of parameter change influence the range of design thicknesses required by each parameter. Therefore, the sensitivity analysis was expanded to examine the combined effects of both the manner in which parameters were included in the thickness function and the actual parameter variations that occur. This combination resulted in 2 measures of practical importance. One relative percentage measure of importance is mathematically shown as follows:

$$RPTC = 100 \times \left| \frac{\partial T}{\partial P_i} dP_i \right| \div \sum_{i=1}^{n} \left| \frac{\partial T}{\partial P_i} dP_i \right| \tag{2}$$

where

$RPTC$ = relative percentage of the thickness change,

$\left| \frac{\partial T}{\partial P_i} dP_i \right|$ = absolute value of the product of the first partial derivative of the thickness function T with respect to the design parameter P_i and the incremental parameter change dP_i, and

$\sum_{i=1}^{n} \left| \frac{\partial T}{\partial P_i} dP_i \right|$ = summation of the absolute values of all parameter products (partial derivative × increment) considered in the sensitivity analysis of a design method.

If appropriate dP_i increments are chosen as representative of actual parameter variations, then the absolute value of the term $\frac{\partial T}{\partial P_i} dP_i$ becomes a realistic measure of the change in thickness caused by a pavement design parameter. The portion of the total thickness change produced by a parameter and calculated as a percentage of the total change provides a relative measure of design parameter importance with respect to the total design process rather than the formulated thickness function alone.

Because the validity of the relative percentage of the thickness change may be questionable as deviations from the mean parameter values become large, a second approach was devised as a corroborating measure of the practical importance of design parameters. If 2 different values are selected for the same parameter, then 2 corresponding thicknesses are determined in the design of a rigid pavement. The difference between these 2 thicknesses is interpreted as the actual change in the design thickness produced by the parameter and its variation if the 2 parameter values are indicative of realistic parameter variations normally encountered. Based on this uncomplicated concept, a thickness change caused by a variation in one parameter can be expressed as a percentage of the sum of all changes that are similarly calculated for each design parameter. This second measure of practical importance is summarized by the following equation:

$$RPATC = 100 \times \left| \Delta T_{P_i} \right| \div \sum_{i=1}^{n} \left| \Delta T_{P_i} \right| \tag{3}$$

where

$$\mathrm{RPATC} = \text{relative percentage of the actual thickness change,}$$
$$|\Delta T_{P_i}| = \text{absolute change in thickness due to the } P_i \text{ parameter, and}$$
$$\sum_{i=1}^{n} |\Delta T_{P_i}| = \text{sum of all the absolute thickness changes produced by all parameters in the design method under consideration.}$$

Because very little difference was observed between the 2 measures of practical importance, only evaluations for the partial differential method are presented in this report of the sensitivity analysis of 3 rigid pavement design methods. The increment of parameter change was always selected to approximate 1 standard deviation of the "population" for each design parameter.

RESULTS

The techniques of sensitivity analysis that were developed in the preceding section were applied to the design methods of AASHO, Corps of Engineers, and PCA. Both theoretical and practical importance measures were quantified to demonstrate the significance of each design parameter in determining the resultant rigid pavement thickness.

AASHO

The sensitivity analysis of the AASHO rigid design method used the following equation (1):

$$\log W = 7.35 \log (D + 1) - 0.06 + \frac{\log \dfrac{c_0 - p}{c_0 - 1.5}}{1 + \dfrac{1.624 \times 10^7}{(D + 1)^{8.46}}} + (4.22 - 0.32p) \log \left[\frac{f_t \,(D^{0.75} - 1.132)}{690 \left(D^{0.75} - \dfrac{18.416k^{0.25}}{E^{0.25}} \right)} \right] \quad (4)$$

where

W = total number of 18-kip single-axle load applications;
D = concrete pavement thickness, in.;
c_0 = initial serviceability index;
p = terminal serviceability index;
f_t = 0.75 × modulus of rupture of concrete = working flexural strength, lb/in.^{2};
k = modulus of subgrade reaction, lb/in.^{3}; and
E = modulus of elasticity, lb/in.^{2}.

Each of the variables in the above equation was investigated in the sensitivity analysis to evaluate its relative influence on the design thickness D. Although importance measures were calculated for many combinations of design parameter values, a summary of the sensitivity analysis is presented only for likely combinations in this report.

Summary plots of theoretical parameter importance are shown in Figure 1 for terminal serviceability indexes of 2.0 and 2.5. The terminal and initial serviceability indexes are theoretically the most important design parameters in the formulated thickness function. In addition, design conditions that are reflected by subgrades and concretes of different strengths and traffic loads of different magnitudes have little influence on the theoretical importance of the AASHO parameters.

If a subgrade modulus of 100 lb/in.^{2}, a concrete working flexural strength of 450 lb/in.^{2}, a terminal serviceability index of 2.0, an initial serviceability index of 4.0, and a 4-lane highway operating at a volume level of 1,200 passenger cars per hour per lane with 20 percent trucks are regarded as a typical design situation, then the approximate corresponding relative theoretical parameter importances are 77 percent for the terminal serviceability index, 21 percent for the initial serviceability index, and 2 percent for the concrete, the subgrade, and the traffic parameters. If the terminal serviceability index is increased from 2.0 to 2.5 and no changes are made in the other design parameters, then 65, 33, and 2 percent are respectively the relative theoretical parameter importances for the terminal serviceability index, the initial serviceability index, and the combination of the concrete, the subgrade, and the traffic design parameters. Thus, approximately 98 percent of the pavement thickness is based on the 2 measures of pavement serviceability when the sensitivity analysis of the AASHO rigid design method is performed to ascertain the relative theoretical importance of the various design parameters.

Practical measures of relative importance, which provide a more realistic determination of parameter importance in the design process by accounting for the actual amount of variation that occurs in each factor, are shown in Figure 2 for the AASHO rigid design method. Ranges of practical importance that represent various percentages of trucks and traffic volumes are shown by short horizontal lines and extended curves respectively. For parameters of lesser importance, the short lines or extended curves are not distinguishable, and a single line adequately represents the range of these practical measures. The 5 short lines identify parameter importance for a traffic stream composed of 10, 20, 30, 40, and 70 percent trucks, and the 3 extended lines describe practical importance for a highway that carries the equivalent of 1,200, 1,500, and 2,000 passenger cars per hour per lane. Because greater traffic factor values imply higher volumes of traffic or a larger percentage of trucks, the conditions represented by each line can easily be ascertained.

Because each plot demonstrates the controlling importance of the AASHO traffic parameter, the total equivalent 18-kip single-axle load was concluded to be the most important parameter influencing the design objective. The concrete modulus of rupture ranked second in practical importance. However, as the weight and frequency of vehicular traffic increase, the traffic parameter becomes less important, and the concrete flexural strength and the terminal serviceability index assume an increasing importance in the determination of rigid pavement thicknesses. If the same typical design situation as appraised for the theoretical parameter importance is again assumed for a pragmatic evaluation of the AASHO design factors, then the relative practical parameter importances are 60 percent for the traffic parameter, 20 percent for the flexural strength parameter, 9 percent for the modulus of subgrade reaction, 9 percent for the terminal serviceability index, and 2 percent for the initial serviceability index. Although the AASHO traffic parameter shows the most significant increase in practical importance as compared to the theoretical measure in which it had no influence, the serviceability indexes exhibit the most prominent decrease. In both sensitivity analyses according to the theoretical and the practical importance of each design parameter, the measure of subgrade support is relatively insignificant in the thickness determination of rigid pavements.

Corps of Engineers

Because the rigid design method of the Corps of Engineers was based substantially on the plotted empirical relations (4), the design chart shown in Figure 3 was selected as the best available model for the sensitivity analysis. The few mathematical equations pertaining to this design technique did not express the relations used to prepare the design chart and, hence, could not be used to yield the necessary information that could be obtained by graphical interpretation of the design chart.

In the evaluation of the theoretical importance of the various parameters, a graphically determined first-order partial derivative of thickness with respect to each design parameter was expressed as a percentage of the sum of all partial derivatives for a wide range of parameter-value combinations. The theoretical importance measures are shown in Figure 4 for 2 common design conditions. An overwhelming theoretical

Figure 1. Relative theoretical importance of AASHO rigid design parameters.

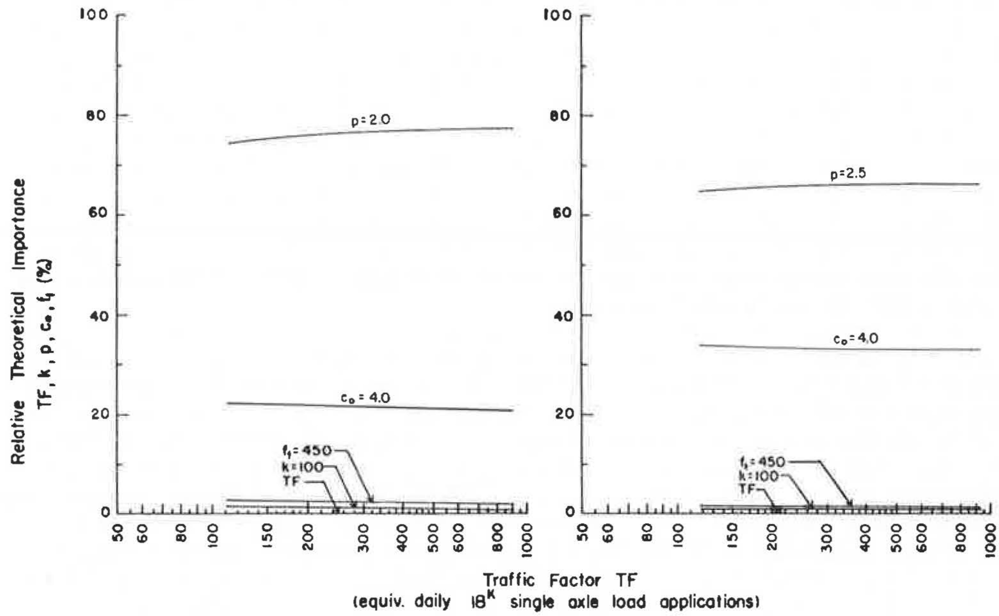


Figure 2. Relative practical importance of AASHO rigid design parameters for total differential approach.

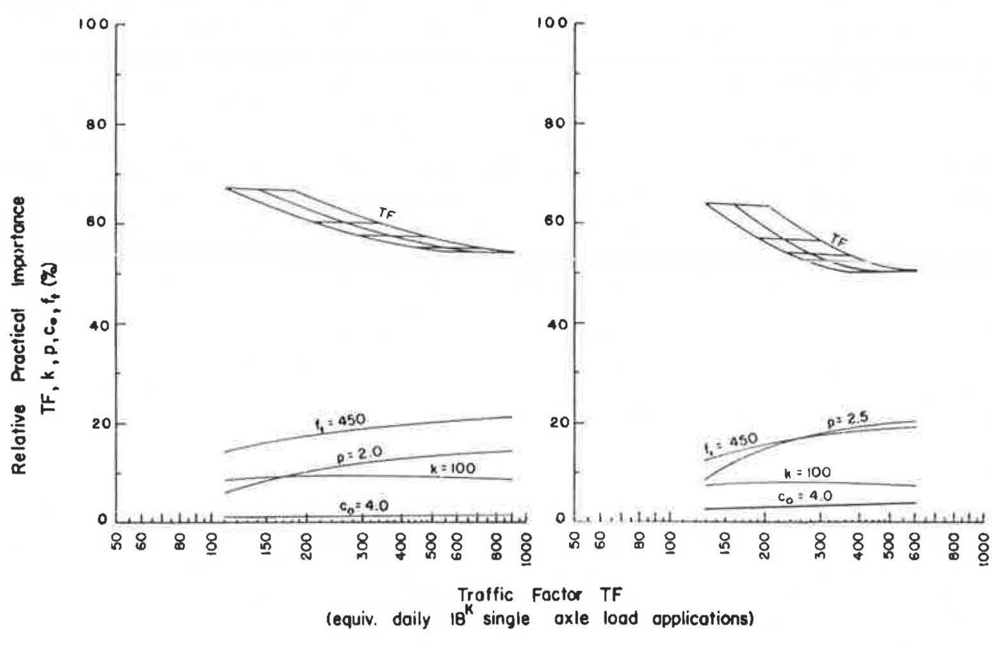


Figure 3. Rigid design chart of Corps of Engineers.

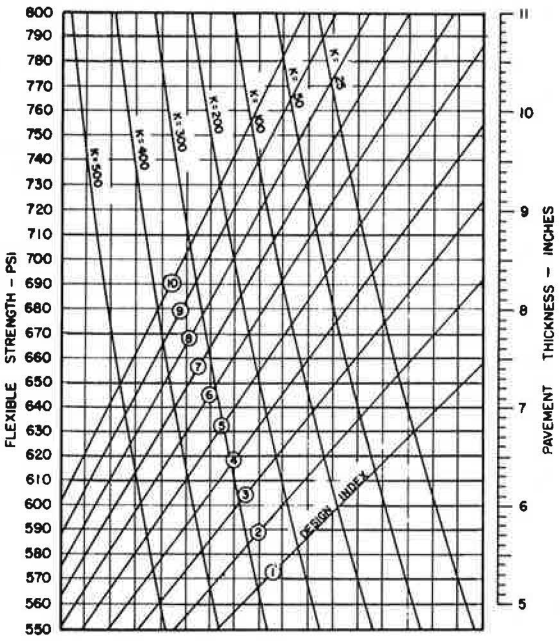
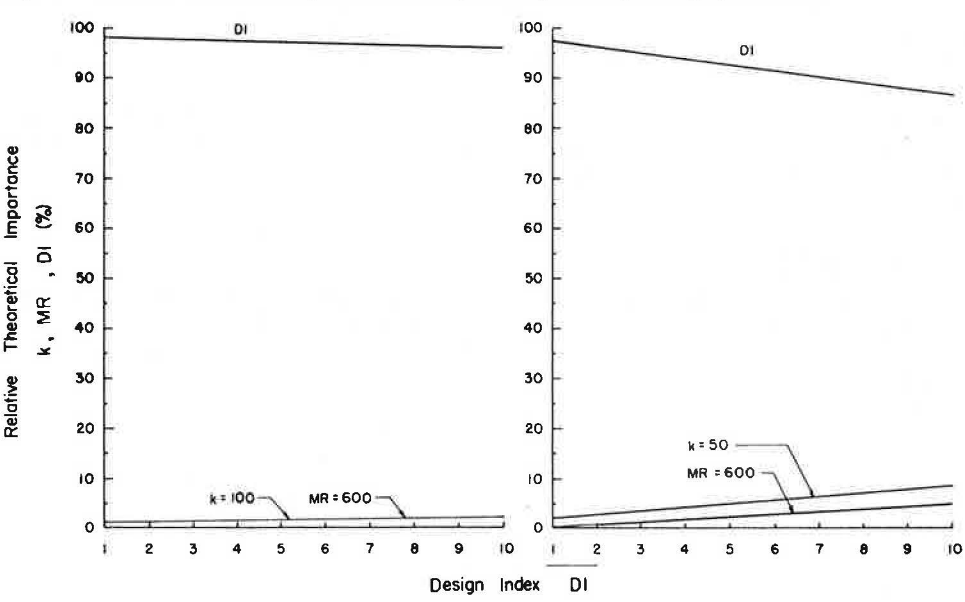


Figure 4. Relative theoretical importance of Corps of Engineers rigid design parameters.



importance is evidenced for the design index (traffic) parameter for most design situations. Either the subgrade modulus or the modulus of rupture accounts for no more than 10 percent of the relative rate of thickness change, and the design index accounts for no less than 85 percent of that measure of importance. If actual parameter variations are neglected, the Corps of Engineers rigid thickness requirements could essentially be quantified by a traffic load estimation alone.

Parameter variations, however, are an integral part of the design process and were considered in developing the relative practical measures of parameter importance. To predict representative variations in the Corps of Engineers rigid traffic parameter, we assumed that the design index was directly and linearly proportional to the traffic parameter used in the California Division of Highways stabilometer design method for flexible pavements. Thus, readily determined variations in the California stabilometer traffic index were translated into similar approximate variations of the Corps of Engineers rigid design index parameter. This method of estimating Corps of Engineers rigid design index variations does not enable average daily traffic or percentage of trucks to influence directly the increments of change in this design parameter. Design index values greater than 5 do not apply to the usual pneumatic-tired vehicular traffic and were neglected in the practical importance measures.

Because the practical importances of the design parameters were nearly identical for both the "portions-of-the-total-differential" and the "relative-thickness-changes" approaches, only values calculated by the former method are shown in Figure 5 for subgrade reaction values of 100 and 50. For the average conditions described by a design index of 4.0 (k = 100), the relative practical parameter importance is approximately 38 percent for the modulus of rupture, 42 percent for the subgrade modulus, and 20 percent for the design index. As the design index value decreases, there is a significant decrease in its relative importance. A decrease in the modulus of subgrade reaction, which implies weaker supporting soils, indicates a slight increase in the importance of that parameter with a corresponding decrease in the importance of the design index. Stronger concretes as reflected by increases in the value of the modulus of rupture produce a slight decrease in the importance of both the design index and the modulus of rupture and an increase in the importance of the subgrade reaction.

For the Corps of Engineers rigid design method, the modulus of subgrade reaction and the modulus of rupture are the important design variables. The traffic parameter is ranked third for the assumptions used in developing the practical importance measures.

Portland Cement Association

Although further development of thickness models for the sensitivity analysis of the AASHO and the Corps of Engineers rigid processes was not necessary, the following mathematical model of the total PCA rigid design process (3) was formulated in this research investigation:

$$\log\left[ADT\ (PTT)\ AT_i\right] = \frac{20.24P_i}{(MR)\ D^2}\left[1.0 - \frac{0.2034(a_i)^{0.500}(k/D^3)^{0.125}}{0.925 + 0.0091(A_i)(k/D^3)^{0.250}}\right] - 12.0 \quad (5)$$

where

- ADT = average daily traffic in both directions, vehicles per day;
- PTT = percentage of total traffic that is trucks;
- AT_i = axles per 1,000 trucks for axle load class i;
- P_i = design load equal to the midpoint of axle load class i, lb;
- MR = modulus of rupture, lb/in.^{2};
- D = pavement thickness, in.;
- a_i = radius of contact, in.; and
- k = modulus of subgrade reaction, lb/in.^{2}

This design model relates the modulus of subgrade reaction, the modulus of rupture, and the 3 factors characterizing traffic loads to the required thickness for a 125 percent fatigue resistance as the failure criterion. In the development of this equation for the sensitivity analysis, reasonable assumptions were made in regard to the lateral distribution of traffic, the axle load allocations among vehicles in the traffic stream, and the design life of the facility.

The relative theoretical importance of each design parameter as indicated by the first partial derivative of the thickness function is shown in Figure 6 for modulus of rupture values of 600 and 500 $lb/in.^2$. To facilitate the presentation of the relative importance measures, appropriate California stabilometer traffic index values were selected to summarize the traffic parameter for the 3 PCA factors that describe traffic conditions. For "average" conditions (MR = 600 $lb/in.^2$, k = 100 $lb/in.^2$, and TI = 12), the relative theoretical importance measures are 66 percent for the modulus of rupture, 31 percent for the subgrade modulus, 2 percent for the axles per 1,000 trucks parameter, 1 percent for the percentage of trucks, and negligible importance for the average daily traffic.

The same combinations of design parameter values were also used in the evaluation of practical parameter importance as shown in Figure 7 (MR = 600). For the same average conditions, the relative practical importances are 60 percent for the modulus of rupture, 25 percent for the subgrade modulus, 15 percent for the average daily traffic, and a negligible amount for the other 2 traffic factors. This practical measure of importance indicates a remarkable increase in the importance of the average daily traffic as a design parameter, but little change is noted from the theoretical measures for the other parameters. In this comparison between the theoretical and the practical measures of relative importance, the average daily traffic parameter increased from 0 to 15 percent, and the relative importances of the modulus of rupture for concrete and the subgrade modulus were each reduced on the average by 6 percent. The remaining 3 percent is accounted for by the loss of importance in the parameters of the axles per 1,000 trucks and the percentage of trucks. Therefore, the modulus of subgrade reaction and the modulus of rupture are the more important parameters in the PCA rigid design process.

SUMMARY OF RESULTS AND CONCLUSIONS

The AASHO, Corps of Engineers, and Portland Cement Association design methods were analyzed to identify and examine the relative effects on thickness of the various factors considered in these pavement design processes. A modeling of the design technique, a measure of relative theoretical parameter importance, and 2 measures of relative practical parameter importance were employed in the investigation of these rigid pavement design methods. Although the design objective of an adequate highway pavement to serve the imposed physical and subjective demands is common to all pavement design techniques, the factors considered and their manner of employment are particular to each design technique. Subgrade characteristics, traffic loads, concrete flexural strength, and performance criteria are generally regarded as the primary factors affecting rigid pavement design.

After a model of each technique that was adequate for the sensitivity analysis was formulated, a quantitative evaluation of the theoretical influences of major factors on thickness delimited the inconsistencies among rigid design techniques. Although the Corps of Engineers rigid traffic parameter is of considerable theoretical importance, the AASHO and the PCA rigid traffic parameters have a negligible influence in the formulated thickness functions. In a similar manner, the design parameters of soil support and pavement material are theoretically important elements in the design of a rigid pavement by the PCA rigid process but unimportant in the AASHO and the Corps of Engineers rigid methods. A numerical summarization of the theoretical importance of the various design parameters is given in Table 1 for each design method.

Dissimilarities were also evident among the 3 rigid pavement design methods as a result of the sensitivity analysis for practical importance. The relative values of practical importance are given in Table 1 for the design parameters that are appropriate

Figure 5. Relative practical importance of Corps of Engineers rigid design parameters.

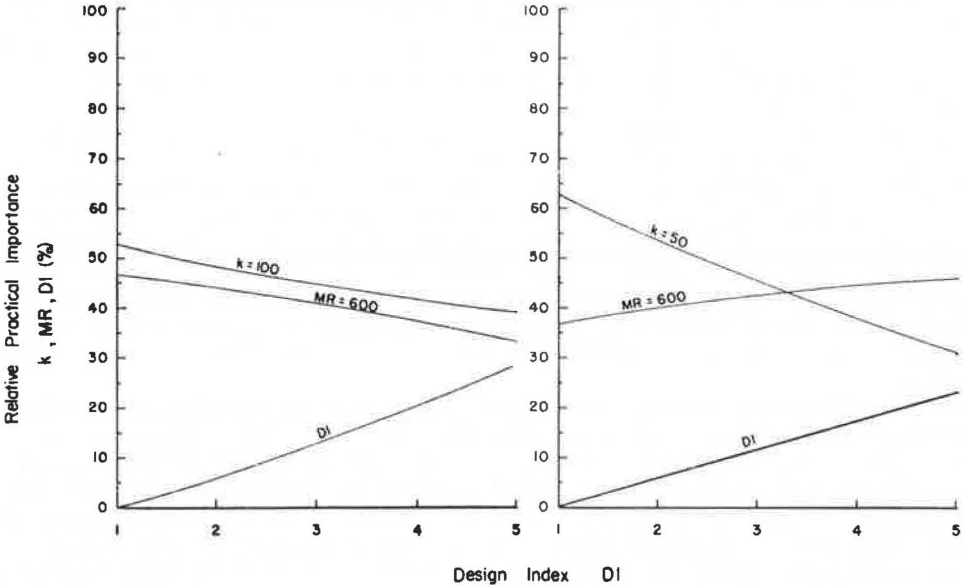


Figure 6. Relative theoretical importance of PCA rigid design parameters.

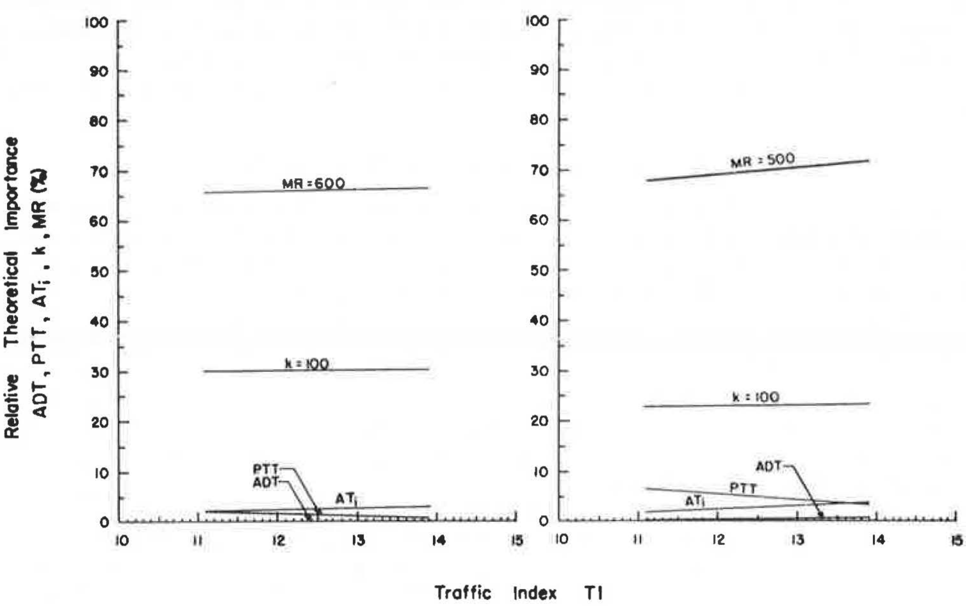


Figure 7. Relative practical importance of PCA rigid design parameters.

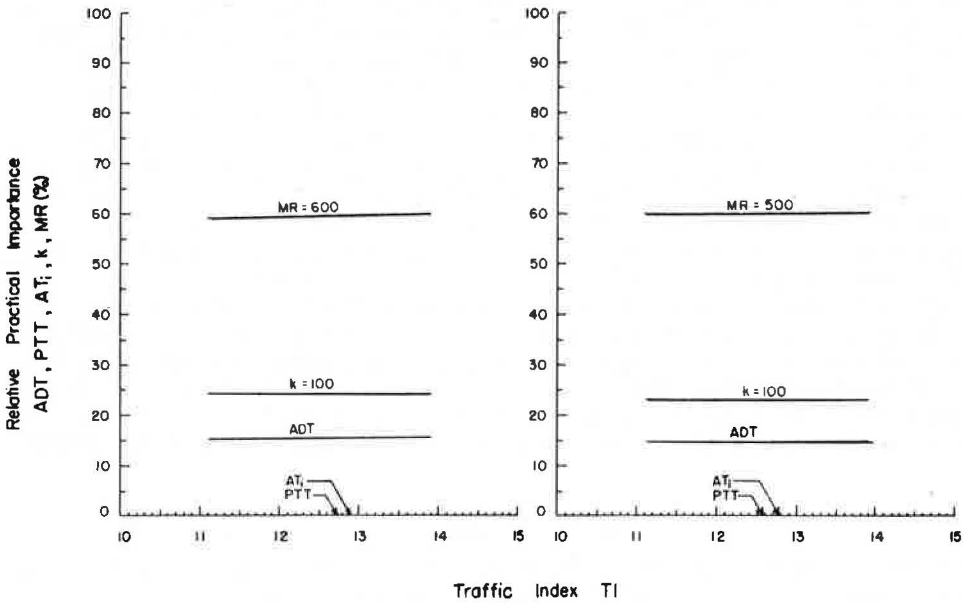


Table 1. Parameter importance in percent for average design conditions.

Design Method	Importance Measures	Traffic[a]	Supporting Soil k	Pavement Material[b]	Initial Performance c_o	Terminal Performance p
AASHO	Theoretical	0	1	1	21	77
	Practical	60	9	20	2	9
Corps of Engineers	Theoretical	98	1	1	—	—
	Practical	20	42	38	—	—
PCA	Theoretical	0, 1, 2	31	66	—	—
	Practical	15, 0, 0	25	60	—	—

[a]TF for AASHO; DI for Corps of Engineers; and ADT, PTT, and AT_i respectively for PCA.
[b]f_t for AASHO and MR for Corps of Engineers and PCA.

to each pavement design procedure. Although the traffic parameter is highly important in the AASHO rigid design method, the input of traffic data provides only minor practical importance in the determination of rigid pavement thicknesses by the Corps of Engineers and the PCA design processes. On the other hand, soil support is the most important design parameter in the Corps of Engineers rigid design method, although the strength of the pavement material occupies a slightly less degree of practical importance. The strength characteristics of concrete are highly important in the PCA rigid design procedure. However, the pavement material provides less input in the thickness determination for both the Corps of Engineers rigid and the AASHO rigid design methods. Soil support does account for some practical importance in the PCA rigid design procedure, but this design parameter has little quantitative impact on pavement thickness when the AASHO rigid design method is selected.

This sensitivity analysis of the physical variables and the subjective factors that affect thickness and expected pavement performance provides the design engineer with greater insight into the decision-making process of accomplishing the structural design of rigid pavements. The effect of actual deviations associated with these parameter design values identifies those phases of the design process that require closer attention and study and indicates those areas where design information is exceedingly precise.

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SELECTION OF OPTIMAL PAVEMENT DESIGNS CONSIDERING RELIABILITY, PERFORMANCE, AND COSTS

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This paper deals with the overall problem of selecting the best or optimum pavement design strategy for a given project situation. Many alternative design strategies for any given project are possible because of the many possible combinations of alternative materials, layer thicknesses, and future maintenance and rehabilitation options. In addition, imposed in any given design situation are several constraints, such as maximum funds available for initial construction, limiting policies of a particular agency, and future availability of maintenance and rehabilitation funds. Each of the possible design strategies has associated costs (and benefits), performance, and reliability—3 important judgment factors that may be used to help select the optimal design strategy. This paper discusses the derivation of alternative design strategies, their associated costs, performance, and reliability, and the use of the total cost method of economic analysis to select an optimal design strategy within the various design constraints. The optimum design is defined here as the design that has the minimum total cost (including construction and future maintenance, rehabilitation, salvage, and user costs) and is within the various constraints that are specified by the designer. Certain practical constraints such as a minimum acceptable design reliability level or funding limitations may modify the selection of the optimum design strategy somewhat.

•SELECTION of the optimum or best pavement design strategy for an airport or highway pavement is of vital concern to the engineer. Factors such as pavement reliability, performance, and costs and their interrelations must be considered by the engineer in selecting the optimum design. The purpose of this paper is to outline a methodology and to make recommendations for selecting an optimal design strategy.

The recent development and implementation of computerized pavement design systems have focused attention on the many possible alternative designs that exist for any given project. Usually several material alternatives, many possible combinations of layer thicknesses, and many alternative future maintenance and overlay policies can combine to give a large number of possible alternative design strategies. There are also the inherent uncertainty in traffic prediction and the many inaccuracies involved in the design models and in the accurate estimation of design inputs for predicting future performance. Historically these variations have been considered in design by the use of safety factors or other arbitrary decisions based on experience. However, for more uniform overall design, a reliability concept is needed.

How can a pavement engineer sort out the variables, analyze the problem, and select the best design for a given project? What decision criteria should be used in selecting

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an optimal design? The scarcity of highway funds, materials, and fuel and the desire for adequate pavement performance make the need to select optimum designs greater than ever before. The use of basic considerations of pavement costs, reliability, and performance along with engineering judgment and a systems approach provides the framework for such selection procedures. Considerable experience in this area has been gained in developing and implementing flexible and rigid pavement design systems for the Texas Highway Department (3, 9, 10).

In this paper the nature of the problem is considered, and then pavement performance, reliability, and costs are discussed. Finally, the concepts needed to determine the optimum design strategy are presented.

ALTERNATIVE DESIGN STRATEGIES

A complete pavement design strategy includes not only the initial pavement structure to be constructed but also any future rehabilitation needs (i.e., maintenance, overlays, and seal coats) and the general traffic-handling methods to be used during rehabilitation. Each strategy is analyzed in terms of its predicted performance and estimated costs throughout the analysis period.

Consider the actual freeway pavement design situation given in Tables 1 and 2. These specific inputs were used in the Texas flexible pavement design system for the example problem considered.

1. The surfacing is to be hot-mixed asphalt concrete, the base is to be an asphalt-stabilized gravel, the first subbase layer if used can be crushed limestone or lime-stabilized material, and the second subbase layer if used can be untreated granular material or a layer of lime-stabilized subgrade soil.
2. Based on previous experience, the surface thickness is set at 1.5 in. (38.1 mm), the asphalt-stabilized base thickness is set from 5 to 8 in. (127.0 to 203.2 mm), and the subbase thickness if selected by the program is set from 5 to 10 in. (127.0 to 245.0 mm).
3. The pavement design may or may not provide for future asphalt concrete overlays for stage construction. Minimum time to the first overlay is set at 4 years with 6 years minimum between overlays. Routine maintenance such as spot patching and crack filling will be necessary.
4. Various safety factors could be applied to allow design at varying levels of reliability.
5. Several ways of handling traffic during overlay operations are available with probable differences in user delay costs, depending on traffic volume magnitude.

This design problem was run through the Texas flexible pavement design system, and the results illustrate the many possible design strategies that can occur through available combinations of materials, thicknesses, maintenance policies, traffic-handling methods, and safety factor levels for this typical pavement design. The total number of feasible designs considered was 2,671; several are given in Table 3. Examples of 3 alternative designs are shown in Figure 1. Figure 1a and 1b show designs with a reliability level of 50 percent, and Figure 1c shows a design with a reliability level of 95 percent. The problem is to select the optimum design strategy from among the many possible strategies of which these are only illustrative. The criteria to be used to select the best or optimum design strategy must be determined. Three important judgment or evaluation criteria to be used in the selection are pavement performance, reliability, and total costs. Engineering experience must be included in the evaluation process mainly because of inadequacies in the predictive capability of the pavement design procedures now available.

Thus a pavement design strategy may be considered in terms of its performance, which gives a measure of the level of serviceability the pavement is giving to the user throughout its design life; its reliability, which gives an index of the probability that the desired level of performance will be achieved; and total costs (and benefits if they can be estimated), which give a monetary value required to provide such a pavement strategy. Each of these factors is briefly described.

Table 1. Design input data for flexible pavement system example problem.

Input	Value
Basic design criteria	
Length of analysis period, years	20.0
Minimum time to first overlay, years	4.0
Minimum time between overlays, years	6.0
Minimum serviceability index P2	3.0
Design confidence level, percent	95
Interest rate or time value of money, percent	7.0
Program controls and constraints	
Number of summary output pages desired, 8 designs/page	3
Number of materials	6
Maximum funds available per yd^2 for initial design, dollars	9.99
Maximum allowed thickness of initial construction, in.	36
Accumulated maximum depth of all overlays (excluding level-up, in.)	8.0
Traffic data	
ADT at beginning of analysis period, vehicles/day	4,260
ADT at end of 20 years, vehicles/day	12,400
One-direction 20-year accumulated number of equivalent 18-kip single-axle loads	3,661,000
Average approach speed to overlay zone, mph	60.0
Average speed through overlay zone (overlay direction), mph	40.0
Average speed through overlay zone (nonoverlay direction), mph	50.0
Proportion of ADT arriving each hour of construction, percent	7.0
Trucks in ADT, percent	11.0
Environment and subgrade	
District temperature constant	30.0
Swelling probability	1.00
Potential vertical rise, in.	5.00
Swelling rate constant	0.15
Subgrade stiffness coefficient	0.24
Construction and maintenance data	
Serviceability index of initial structure	4.2
Serviceability index P1 after an overlay	4.2
Minimum overlay thickness, in.	0.3
Overlay construction time, hours/day	10.0
Asphaltic concrete compacted density, tons/yd^3	1.80
Asphaltic concrete production rate, tons/hour	90.0
Width of each lane, ft	12.0
First-year cost of routine maintenance, dollars/lane-mile	50.00
Incremental increase in maintenance cost per year, dollars/lane-mile	20.00
Detour design for overlays	
Traffic model used during overlaying	3
Total number of lanes of facility	4
Number of open lanes in restricted zone (overlay direction)	1
Number of open lanes in restricted zone (nonoverlay direction)	2
Distance traffic is slowed (overlay direction), miles	2.00
Distance traffic is slowed (nonoverlay direction), miles	0.20
Detour distance around overlay zone, miles	-0.00
Paving materials (Table 2)	

Note: 1 yd^2 = 0.8 m^2, 1 in. = 25.4 mm, 18 kip = 80 kN, 1 mph = 0.04 m/s, 1 ton/yd^3 = 1,329 kg/m^3, 1 ton/hour = 0.025 kg/s, 1 ft = 0.3 m, and 1 mile = 1.6 km.

Table 2. Design input data for paving materials.

Layer	Code	Materials	Cost per Yd^3	Strength Coefficient	Min Depth (in.)	Max Depth (in.)	Salvage (percent)
1	A	Asphalt concrete pavement	13.99	0.96	1.50	1.50	20.00
2	B	Asphalt-stabilized base	15.46	0.90	5.00	8.00	15.00
3	C	Crushed limestone	7.13	0.60	5.00	10.00	75.00
3	D	Lime-stabilized material	6.02	0.55	5.00	10.00	65.00
4	E	Untreated granular material	5.07	0.40	6.00	10.00	50.00
4	F	Lime-stabilized subgrade soil	2.87	0.30	8.00	8.00	0.00

Note: 1 in. = 25.4 mm, and 1 yd^3 = 0.76 m^3.

PAVEMENT PERFORMANCE

A clear definition of pavement performance is essential to the selection of an optimum design. Pavement performance has been defined as a "measure of the accumulated service provided by a pavement, i.e., the adequacy with which a pavement fulfills its purpose" (1). As used in this study, it is more explicitly defined as the integral of the serviceability curve over a specified time period for a section of pavement. Performance thus represents the area beneath the serviceability-time plot. Serviceability is defined as "the ability of a specific section of pavement to serve high-speed, high-volume, mixed (truck and automobile) traffic" (2). The measure of serviceability at any point in time is in terms of a subjectively based panel rating, which can be estimated by some equation that uses objective mechanical measurements. In Texas, this is known as the serviceability index, SI, which is based on the same 0-to-5 scale as used at the AASHO Road Test.

The models that currently exist for predicting the performance of a pavement are only approximate. Also, the serviceability of a pavement varies significantly along a project because of the random nature of distress. Serviceability index values can be determined for consecutive 0.2-mile (0.3-km) sections of pavement for example. A typical distribution of SI values for each 0.2-mile section in both directions along 15 miles (24 km) of highway pavement is shown in Figure 2. The mean SI is 2.7, but it varies from 1.6 to 3.9 along the project. A project average may be computed and plotted with time along with the distribution of SI values to completely characterize the project performance. User costs and rehabilitation costs are directly related to the serviceability (or performance) of a pavement as subsequently discussed.

PAVEMENT RELIABILITY

The following general definition of pavement reliability has been formulated (4): Reliability is the probability that the pavement system will perform its intended function during its design life (time) under the conditions (or environment) encountered during operation. The 4 basic elements involved in this concept of pavement system reliability are probability, performance, time, and environment. Reliability is the probability of success that a system has in performing its function. There are significant variations and uncertainties in prediction associated with all the models in any pavement design system, and, therefore, the chance of success will always be less than 100 percent. The phrase "to perform its function" refers to the actual serviceability time history being as good as or better than predicted during design. Time is an essential element in the definition of reliability because the reliability of a pavement must be considered over a design analysis period. The environmental conditions include the operating circumstances under which the pavement is used. The environment of a pavement greatly affects its life span, its performance, and consequently its reliability. Thus, if the environment changes significantly from that for which the pavement was originally designed, the pavement may not perform with the same reliability as it would have without the change.

An increase in the reliability of a pavement design may be accomplished by applying safety factors to various design parameters or directly to the components of the design strategy, such as pavement thicknesses. For example, the expected design traffic might be increased, or the allowable design subgrade strength might be reduced. This would result in different pavement design strategies that have a greater overall safety factor or higher reliability. The importance of this reliability concept may be illustrated by considering a design model that was derived by using a statistical least squares regression (such as that in the AASHO Interim Guide) and then by using average values of design inputs. The resulting pavement design strategy is only 50 percent reliable with respect to performing as intended. The overall performance safety factor is essentially 1.0 in this situation. The resulting average performance of this design may often not be satisfactory to the user or design engineer, and therefore some types of safety factors need to be applied.

A rational method for applying safety factors to pavement design so that pavement reliability may be estimated has been developed by Darter and Hudson (3). This theory

Table 3. Alternate pavement design strategies for example problem.

Output	Best Design Strategies in Order of Increasing Total Cost					
	1	8	12	18	20	24
Material arrangement	ABD	ABC	ABD	ABD	ABC	ABC
Cost, dollar/yd²						
Initial construction cost	4.07	4.23	4.16	4.06	4.26	4.14
Overlay construction cost	0.57	0.55	0.57	0.72	0.57	0.72
User cost	0.03	0.03	0.03	0.05	0.03	0.05
Routine maintenance cost	0.16	0.16	0.16	0.16	0.16	0.16
Salvage value	-0.35	-0.38	-0.31	-0.31	-0.34	-0.36
Total cost	4.49	4.59	4.62	4.67	4.69	4.71
Number of layers	3	3	3	3	3	3
Layer depth, in.						
D(1)	1.50	1.50	1.50	1.50	1.50	1.50
D(2)	5.00	5.50	6.00	5.75	6.25	5.75
D(3)	8.00	6.50	6.00	6.00	5.00	5.50
Number of performance periods	3	3	3	4	3	4
Performance time, years						
T(1)	5.4	5.5	5.3	5.1	5.3	5.1
T(2)	13.1	13.4	12.7	11.7	12.6	11.9
T(3)	20.9	21.4	20.2	18.5	20.1	18.9
T(4)				24.5		25.0
Overlay policy (including level-up)						
O(1)	1.3	1.3	1.3	1.3	1.3	1.3
O(2)	1.3	1.3	1.3	1.3	1.3	1.3
O(3)				1.3		1.3
Swelling clay loss, units of serviceability index						
SC(1)	0.93	0.94	0.92	0.89	0.92	0.90
SC(2)	0.51	0.51	0.50	0.49	0.50	0.50
SC(3)	0.16	0.16	0.17	0.18	0.17	0.18
SC(4)				0.06		0.06

Note: 1 yd³ = 0.76 m³, and 1 in. = 25.4 mm.

Figure 1. Alternative design strategies.

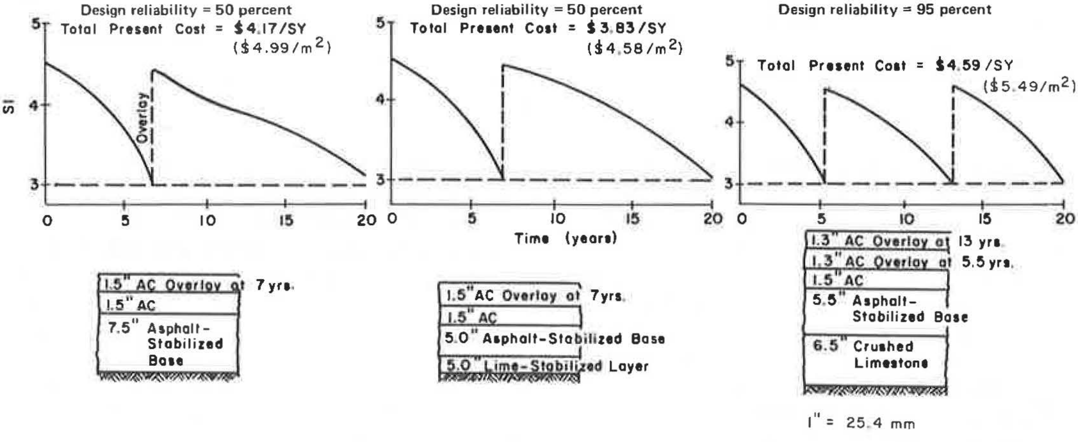
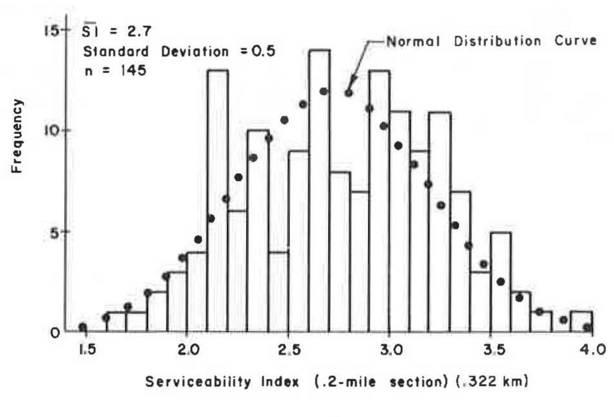


Figure 2. Frequency distribution of SI along in-service highway pavement of uniform design.



was applied to the Texas flexible and the rigid pavement systems (FPS and RPS), which are currently both undergoing implementation by the Texas Highway Department (3, 4, 5, 8).

A brief description of the approach for the application of safety factors to pavement design for determining pavement design reliability is given here, and details can be found in other reports (3, 4). The following conceptual equation illustrates some of the major factors that cause loss of serviceability of a pavement:

Serviceability loss = f(traffic loadings, subgrade shrink-swell, thermal cracking, . . .)

For simplicity of presentation, the probabilistic theory discussed here is limited to the consideration of serviceability loss due to traffic loadings only. The other factors are also important, and the theory should be expanded to consider them in future work.

Stochastic Nature of Design Variables

All pavement design methods that consider loss of serviceability due to repeated traffic loadings (i.e., fatigue) ultimately require the determination of 2 parameters: the prediction of equivalent traffic loads to be applied, n, and the prediction of the equivalent allowable load applications the pavement-subgrade system can withstand, N, to minimum acceptable serviceability. The allowable applications depend on many design factors, such as pavement thickness T, material properties M, and environment E. These are illustrative of the multitude of factors that can affect the multivariate N.

The actual number of applied loads n will depend on many factors, such as average daily traffic A, percentage of trucks t, axle-load distribution L, and equivalency factors F. These factors, estimated for a certain analysis period, are illustrative of those that are involved in the determination of n. To illustrate the process, we can show with appropriate models that N and n are functionally related to the several design variables as follows:

$$N = f(T,\ M,\ E,\ \ldots)$$

$$n = f(A,\ t,\ L,\ F,\ \ldots)$$

In existing design methods, N and n are assumed to be determined precisely by the input variables. In reality, there is considerable variability associated with each design factor. The 3 basic types of variations associated with flexible pavement design parameters may be considered as variation within a design project length, variation between estimated design and actual values, and variation due to lack of accuracy of the design models. The purpose of this study is to develop a method of accounting for this variability in the design process. As a first step, estimates of these variations of the design parameters must be made for in-service highway pavements.

Because all the factors are variable, it therefore follows that f(T, M, E, . . .) and f(A, t, L, F, . . .) are themselves stochastic variables determined by the combined statistical characteristics of the design factors. The N and n have been found to be distributed approximately log normal.

Because N and n are multivariates and stochastic in nature, the variance of each must be determined before the reliability theory can be applied effectively. This may be accomplished by using the partial derivative method (4, 8). The estimates of variance for N and n thus determined may now be used in the next phase where the reliability function is derived.

Reliability Function

Reliability R for pavements is defined here as the probability that N will exceed n.

$$R = P(N > n)$$

This is synonymous with the statement that reliability is the probability that the serviceability level of the pavement will not fall below the minimum acceptable level before the end of the design performance period. If N and n are assumed to be log normally distributed and statistical theory is applied, the following reliability function may be derived:

$$\overline{\log N_R} = \overline{\log n} + Z_R \sqrt{S_{\log N}^{\;2} + S_{\log n}^{\;2}} \qquad (1)$$

where

$\overline{\log N_R}$ = average number of equivalent 18-kip (80-kN) single-axle load applications to be used for design at level of reliability R,

$\overline{\log n}$ = average traffic forecast of equivalent 18-kip (80-kN) single-axle load applications, and,

Z_R = standardized normal deviate from normal distribution tables with mean zero and variance of one for given level of reliability R.

This reliability function may be used either to design a pavement for a specific reliability level or to analyze the reliability of a given pavement.

TOTAL PAVEMENT COSTS

There are several important costs associated with pavement systems. To ascertain the optimum design strategy from an economical standpoint requires that the total costs associated with the pavement facility be determined along with the user costs.

Facility Costs

Total facility costs include those of initial construction, rehabilitation, routine maintenance, and salvage value.

The initial construction cost of the pavement may be considered as the total construction cost of the pavement materials in place. Rehabilitation requirements such as overlays may also be determined by using an in-place construction cost of the overlay material used. The time after initial construction that rehabilitation will be needed and the amount required may be estimated by using various available rehabilitation design models or past pavement performance experience in a particular location. The costs of routine maintenance, such as crack filling, may be estimated from past experience by using maintenance cost records in a given location for a particular type of pavement structure.

There are also the costs associated with the inherent value of the pavement structure materials at the end of the design analysis period. The value of a material at the end of the analysis period may be predicted by estimating a percentage of its original construction value. This will be essentially a negative cost of the total facility cost. Those costs that occur after initial construction must be discounted to the present time by using an appropriate interest rate. These would include maintenance and salvage costs. Each of these costs could be converted to a cost/unit area of pavement. The total facility cost then may be determined as follows:

$$TFC = IC + RHB + RM - SV \qquad (2)$$

where

TFC = present value of total pavement facility cost,

IC = initial construction cost,

RHB = present worth of the sum of rehabilitation costs,

RM = present worth of the sum of routine maintenance costs, and

SV = present worth of the salvage value of the pavement at the end of the analysis period.

Typical details for calculating and estimating each cost factor may be found in a Texas Highway Department manual (5) and a report by Scrivner et al. (6).

User Costs

User costs associated with pavement systems are determined from the increase in cost to users due to pavement maintenance operations and pavement roughness. This results from increased travel time delays, vehicle operation costs, accidents, and user discomfort.

Travel delays may result from both maintenance operations and rough pavements. A procedure to determine travel delay costs and vehicle operating costs due to overlay construction has been developed by Scrivner et al. (6). The model is included in the Texas flexible and rigid pavement design systems. Several traffic-handling techniques were modeled for various types of overlay and highway conditions so that the user delay cost can be estimated. The user delay cost may then be converted to a cost/unit area of pavement by dividing by the area of the pavement overlay operation. Delay in travel time due to a reduction in the serviceability index has been recently estimated by McFarland (7). The reduction in speed was assumed to vary with the serviceability index, type of road, and location (rural and urban) of the pavement. Travel time costs as a function of pavement serviceability index, type of road, and rural-urban location were estimated.

Vehicle operating costs, accident costs, and discomfort costs were also estimated by McFarland (7) as a function of the serviceability index, urban-rural location, and type of road. As the serviceability index decreases, the vehicle operating costs increase in a curvilinear manner. The accident costs were found to increase because of decreased serviceability index. Rural accident costs were determined as a function of the serviceability index and type of road. A final user cost factor related to the serviceability index was discomfort costs. Discomfort costs were determined to be a function of pavement serviceability index, urban-rural location, and type of road.

The total user costs may be approximately estimated from these results in such form as cost/vehicle-mile or cost/unit pavement area as follows:

$$TUC = DLM + DLS + VO + AC + DS \qquad (3)$$

where

TUC = present worth of total user costs,
DLM = vehicle delay costs due to maintenance operations,
DLS = vehicle delay costs due to rough pavement,
VO = costs of vehicle operation caused by rough pavement,
AC = costs of accidents due to rough pavements, and
DS = costs of discomfort due to rough pavement.

This capability makes it possible to estimate, at least approximately, the user benefits (or cost savings to users) for a pavement section during its design analysis period. Therefore, a total pavement cost (facility plus user costs) may be determined for a given design strategy by first expressing these costs in the same units (such as dollar/yd^2, dollar/m^2) and then summing to obtain total pavement costs.

SELECTION OF OPTIMAL DESIGN STRATEGY

Basis for Selection

The pavement design strategy that is selected for construction is usually based on engineering design models, but the selection may also include considerations such as the following: past experience of the designer, including experience with the specific project location and available materials; policies and practices of the particular design agency or "owner" of the project; results previously achieved by the agency using the design procedure; and funds available for initial construction of the project.

The methodology developed in this study for selecting the optimum pavement design strategy is based on the following considerations.

1. There are many feasible design alternatives in any given design situation, and each should be evaluated as a possible candidate for selection. Different project situ-

ations will require different levels of performance and reliability. These levels must be carefully determined. An Interstate highway pavement requires a greater reliability than a farm-to-market road pavement, for example.

2. The costs associated with the pavement user due to travel delays and rough pavements must be considered in an effective pavement alternative design evaluation. The planned or even the unexpected, unplanned maintenance of airport and highway pavements is no longer just the concern of the maintenance foreman.

3. There exists an urgent need to allocate relatively scarce pavement funds and materials in the best manner possible so as to optimize the level of service to the pavement user (including minimizing fuel consumption).

Based on these considerations, the optimum pavement design strategy is defined as the design that provides a satisfactory level of performance to the user at a minimum acceptable level of reliability that the pavement will perform as expected and also has a minimum total cost. The total costs consist of facility and user costs as previously defined. The concept of satisfactory performance level and minimum acceptable reliability level depends on the project characteristics and will subsequently be defined.

Selection of Optimum Design at a Given Reliability Level

A designer usually examines only a few of the many possible design alternatives because of the time involved in manually deriving design strategies, or because of the inability of the design method to consider all alternatives or both. The Texas Highway Department has attempted during the past several years to develop and implement computerized pavement design systems for rigid and flexible pavements. These systems are well documented (3, 6, 9, 10) and provide the engineer with all feasible designs at several levels of reliability. They contain various structural, maintenance, and economic design subsystems that attempt to model the real-world pavement environment during its design life period. The SAMP program developed through NCHRP Project 1-10 is another example of a working system that provides a similar capability and is being prepared for implementation (11).

The alternative designs at a specific level of reliability (or magnitude of applied safety factor) range from strategies with heavy initial construction and low future maintenance to light initial construction with high future maintenance. These alternative designs may be generated on a deterministic basis by pairing all possible combinations of material types and layer thicknesses and then predicting their performance for various maintenance strategies. This process will result in a number of feasible design strategies at a certain reliability level. The facility and user costs may be calculated for each design and the total cost determined. The designs can then be arrayed in order of increasing total cost to facilitate the selection.

An example design situation is given in Tables 1 and 2 for an actual freeway project. The design confidence level or reliability indicated is 95 percent. The inputs include various constraints such as minimum and maximum layer thickness and maximum funds available for initial construction. The program generated 2,671 feasible alternative designs. This large number was mainly due to the several alternative material types, layer thicknesses, and various maintenance strategies. Several alternative output designs are given in Table 3. They are arrayed in terms of increasing total cost during the design analysis period. The total cost ranges from \$4.49 to \$4.71/yd^2 (\$5.37 to \$5.63/m^2) for the first to twenty-fourth strategy. It could be argued that there is no significant difference in the cost of the first several of these designs because of the difficulty in estimating the many costs involved. Therefore, all of the first several design strategies may be considered equal in cost and can be evaluated in terms of their level of effectiveness or performance, the experience of the engineer with the various materials, and perhaps other decision criteria. The most desirable design strategy may then be selected for this given level of reliability (or applied safety factors). This reliability means that the designer can be about 95 percent confident that the pavement will perform as good as or better than predicted. The effect of swelling subgrade soil in reducing the pavement serviceability index is also considered in this case (Table 3).

Optimum Design Strategies at Several Reliability Levels

Pavement design is usually carried out in practice at only one level of reliability by applying a specified safety factor to one or more design inputs. For example, three-fourths of the flexural strength of concrete is taken to be the "working stress" in one rigid pavement design procedure. The consideration of designs at various levels of reliability has been done in several ways in the past, however, but not on a formalized basis. A rational method of application of safety factors to pavement design, which has been implemented into the Texas design system, has been discussed. This approach requires that the magnitude of the uncertainties and variations associated with the design system be quantified. The consideration of design strategies at several levels of reliability is important because of the large differences in factors such as user costs and pavement performance at different reliability levels.

As pavement design reliability increases, pavement performance on the average increases, as shown in Figure 3 for 2 design strategies. The mean expected performance curve is higher for the strategy with greater reliability. The general conceptual relation between pavement reliability and performance for a given project situation is shown in Figure 4. At every reliability level a range of alternative designs exists as has been discussed, and each of these designs exhibits a certain performance. The range of performance of these alternatives is shown in Figure 4.

As the level of reliability increases, facility costs increase and user costs decrease. The increase in facility cost with increased reliability is due to the increased costs of more substantial and higher quality pavements needed to obtain higher performance reliability. In other words, to provide a pavement that will have a greater chance of performing as desired, and, on the average, of showing superior performance will result in higher facility costs. The general relation between pavement reliability and facility costs is shown in Figure 5. Again there exists a range of alternative designs at each reliability level as previously discussed. This possible range is shown by the band width in Figure 5. On the other hand, as reliability increases, the pavement user costs, in general, decrease. This occurs because pavements with greater performance reliabilities are, on the average, smoother and require less maintenance and rehabilitation, which results in decreased user delay, accident, vehicle operation, and discomfort costs. They result in greater benefits to the user.

The various interrelations that have been discussed can be summarized by considering the plot shown in Figure 6. This total cost plot was developed by summing ordinates of user and facility costs for each level of reliability. The shape can be logically explained by using 3 hypothetical design strategies, i, j, and k, for a given project, as is indicated on the plot. Strategy i represents a design at a low level of reliability or low safety factor (not less than 50 percent, however). Such a strategy would have a small chance of performing as expected and would on the average exhibit rather low SI throughout its design life and have associated with it several unexpected pavement failures that would require rehabilitation repairs. The facility costs may be relatively low, but user costs would be relatively high because of a low serviceability level of the pavement during its design life. This results in high user delay and vehicle operation costs due to excessively rough pavements.

Strategy k represents a pavement design at a relatively high level of reliability where the corresponding facility costs are very high and user costs are low because of a high performance level of the pavement. This strategy represents very heavy initial construction and minimal or no maintenance.

Strategy j represents a pavement somewhere between the extremes of i and k. This design strategy represents a design that has facility and user costs that combine to give an overall minimum total cost. The level of performance and reliability expected is between that of i and k.

If the total cost method of economic analysis is used, the level of facility cost should be increased until the total cost begins to increase. The point of minimum total cost as shown in Figure 6 represents the optimum design reliability from an economic standpoint. Because the decreases in user cost are the benefits that would be used in the benefit-cost method of analysis, the total cost method gives essentially the same level.

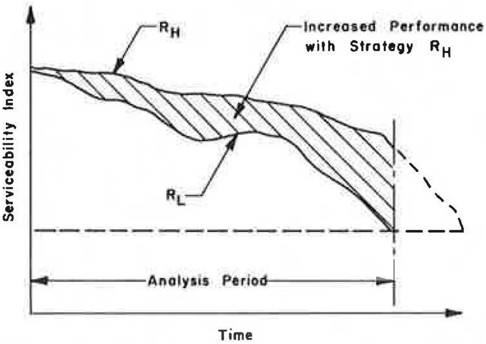


Figure 3. Mean expected performance curves for design strategy with relatively high reliability R_H and low reliability R_L.

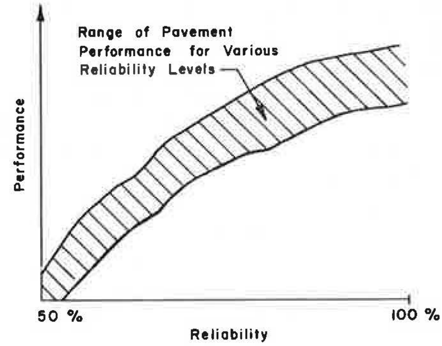


Figure 4. Conceptual relation of reliability and performance.

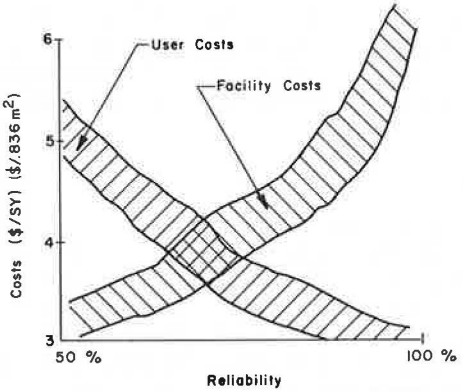


Figure 5. Relation between pavement costs and reliability.

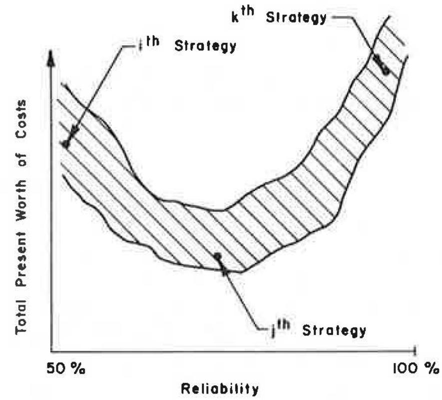


Figure 6. Relation between total pavement costs (user plus facility) and pavement reliability.

Table 4. Projects and selected design reliability level using FPS-11 program.

Project	Highway	Functional Classification[a]	ADT	Equivalent 18-kip Single-Axle Loads	Selective Reliability Level (percent)
1	FM-2625	Minor collector	135[b]-260[c]	30,000	95
2	FM-1840	Minor collector	730-1,215	289,000	95
3	US-70 and US-84	Minor arterial	955-1,950	658,000	95
4	SH-300	Major collector	1,100-1,850	637,400	99
5	SH-24	Minor arterial	1,600-3,200	800,000	99
6	US-84	Principal arterial	1,670-2,750	1,069,000	99
7	US-271	Minor arterial	1,350-2,150	1,450,000	99
8	SH-71	Principal arterial	2,800-4,900	1,562,000	99
9	US-290	Principal arterial	2,130-6,200	3,661,000	99
10	Loop-289	Principal arterial	2,725-16,400	2,840,000	99.99
11	SH-360	Principal arterial	6,800-15,100	4,657,000	99.99
12	Loop-1	Principal arterial	19,660-32,380	6,894,000	99.99

Note: 18 kip = 80 kN.
[a]From ref. 12. [b]Initial 1 direction. [c]End 1 direction.

The reliability associated with design strategy j represents the level that would generally give a minimum total cost for the project and therefore would give the highest benefit-cost ratio. However, other factors that must be evaluated in practice are performance and reliability. Does the expected performance curve give an adequate level of service to the user? This must be judged by the engineer from previous experience and from the magnitude of associated user costs. The variations in serviceability that are shown in Figure 1 must be considered in making this evaluation. Considerable research is needed in this area to better define user requirements. The other factor, which is perhaps easier to quantify, is the minimum practical level of design reliability.

An approximate estimate of the minimum level of reliability for Texas conditions was obtained by developing a series of flexible pavement designs for 12 projects, ranging from farm-to-market roads to urban freeways; the Texas FPS design system was used. Designs were made for each project at 5 levels of reliability, ranging from 50 to 99.99 percent. Experienced engineers then selected the design strategy that they considered adequate for each project. Table 4 gives the projects and levels of reliability selected. The reliability levels shown should be considered only as approximate. The selected design reliability increases with the function or type of highway pavement being designed and the traffic volumes and equivalent load applications.

Therefore, it may be concluded that the minimum level of design reliability increases with the magnitude of possible consequences of failure. These consequences may involve adverse public reaction due to rough pavements or to a new pavement requiring, soon after construction, major rehabilitation that would increase with traffic volume. User delay, accident, or discomfort costs may also be considered as an index of the consequences of failure. The consequences of failure are greater for a higher type of highway, and therefore the minimum design reliability level is greater. Tentative minimum levels of design reliability have been determined for the Texas pavement design system and are given in the manual (5).

SUMMARY

A methodology for determining optimal pavement design strategies is outlined. An optimal design strategy is defined as the design that gives a satisfactory level of performance to the user at a minimum acceptable level of reliability and also that has a minimum total cost. To select an optimal design, one should generate alternative designs at various levels of reliability and determine their associated costs and performance. An optimal design can be selected from these alternatives by using a methodology such as that described here. Engineering judgment and experience play an important part in this last selection process, as discussed.

Even though the designer is somewhat restricted by factors such as inadequate design procedures, agency design policies, and lack of control over maintenance operations, the state of the art of pavement design has progressed to the point that pavement costs (and benefits), performance, and reliability can be roughly estimated and used to assist in selecting optimal pavement design strategies. Considerations such as those presented here will, it is hoped, lead to improved practices that will result in better allocation of scarce pavement funds and in improved service to the user.

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