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EVALUATION OF PARAMETERS
SIGNIFICANTLY INFLUENCING THE
PERFORMANCE OF CONTINUOUSLY
REINFORCED CONCRETE PAVEMENTS

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Technical Paper

EVALUATION OF PARAMETERS SIGNIFICANTLY INFLUENCING THE
PERFORMANCE OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

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TO: J. F. McLaughlin, Director
Joint Highway Research Project

May 1, 1974

FROM: H. L. Michael, Associate Director
Joint Highway Research Project

Project: C-36-52J

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The attached Technical Paper "Evaluation of Parameters Significantly Influencing the Performance of Continuously Reinforced Concrete Pavements" has been authored by Messrs. Asif Faiz and E. J. Yoder. The Paper is from information resulting from the JHRP research study in progress on CRC pavements in Indiana.

The Paper was presented at the recent Annual Meeting of American Concrete Institute in San Francisco. It is planned for publication in the Proceedings of that meeting. In conformance with policy this Paper is presented to the Board for approval of such publication.

Respectfully submitted,

Harold L. Michael

Harold L. Michael
Associate Director

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EVALUATION OF PARAMETERS SIGNIFICANTLY INFLUENCING THE PERFORMANCE OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

SYNOPSIS

This paper reports a portion of findings obtained from a continuing study of CRC pavements in Indiana. Results of an earlier statewide condition survey showed that subbase type, methods of fabrication and placement of steel reinforcement, concrete slump and traffic were significant contributors to CRC pavement performance. It was indicated that method of paving (slipformed vs side-formed) had little effect on pavement performance, while depressed steel resulted in better pavement condition than steel pre-set on chairs. Based on these results, a field investigation of CRC pavements was conducted. The purpose of this study was to evaluate, by means of physical tests, parameters that were found to contribute significantly to CRCP performance in the condition survey.

The results reported in this paper were obtained from a comparative statistical analysis of properties of failed locations with good locations within test sections showing poor performance. The analysis was based on data obtained from 15 CRCP test sections showing significant distress as indicated by a breakup or a patch. Test locations in good condition had subbases with higher stability as evaluated by a static penetrometer test. Pavement failures were observed

to be correlated with relatively low bulk density and modulus of elasticity of concrete. No significant difference in splitting tensile strength of concrete was indicated between good and failed locations. Relative to uniformity of concrete properties above and below the steel reinforcement, no significant difference was shown in the analysis. Higher pavement deflections, wider crack widths and non-uniform crack patterns were associated with failed pavement condition.

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EVALUATION OF PARAMETERS SIGNIFICANTLY INFLUENCING THE PERFORMANCE OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

INTRODUCTION

The use of continuously reinforced concrete pavement (CRCP) dates back to 1938, when an experimental CRC pavement was built on US-40 near Stilesville, Indiana. In recent years, the use of this type of pavement construction has increased considerably. By the end of 1971, 696 miles (1114 km) of equivalent two-lane CRC pavement had been constructed in Indiana. Most of the CRC pavements in Indiana are nine inches (23 cm) thick (primarily on the Interstate System), although some have been constructed seven (18 cm) and eight inches (20 cm) in thickness. The percentage of steel used has been 0.6 percent of the cross-sectional area, irrespective of the other design factors. Generally non-stabilized granular subbases have been used under the pavement, although in recent years the trend has been towards the use of bituminous stabilized subbases.

In 1972, a continuing study of the performance of CRC pavements was initiated by the Joint Highway Research Project at Purdue University. The objective of this study is to evaluate and recommend design and construction techniques that would result in better performance of continuously reinforced concrete pavements.

The first phase of the study involved a condition survey of CRC pavements. The initial activity consisted of a survey of Interstate Highway I-65 to determine the causes of distress that had been noted on certain portions of this road. This

survey indicated some trends relating to the performance of CRC pavements. In order to obtain more conclusive results, it was considered necessary to evaluate a wider range of design and construction factors. Consequently a statewide condition survey of CRC pavements in Indiana was made to study the factors influencing CRCP performance.

The condition survey (5), was statistically designed to include factors considered to be possible contributors to the performance of CRC pavement. The resulting data were analyzed by means of a weighted least squares analysis of variance procedure. The unit of evaluation was a 5000 ft (1524 m) length of pavement. The measures of evaluation were number of failures, number of spalled cracks, the extent of random cracking, parallel cracks with a crack interval of less than 30 in. (76 cm) and the extent of pavement pumping observed per 5000 ft. (1524 m) length of pavement.

The results of the statistical analysis indicated that subbase type, the 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 as compared with the use of tied bar mats. Depressed steel resulted in better pavement condition than the use of steel preset on chairs. The data showed little difference between the condition of pavements that were slipformed as compared to those which were side-formed. Relative to good performance, an optimum range of

concrete slump between 2.0 to 2.5 in. (5.0 to 6.5 cm) was indicated. It was further shown that distress of CRC pavements was related to traffic. Most of the pumping was observed on pavements with gravel subbases though some pumping was also indicated where bituminous stabilized and crushed stone subbases were used.

The second phase of the study consisted of a field investigation of CRC pavements in Indiana. The purpose of this investigation was to evaluate, by means of physical tests, the parameters that were found to contribute significantly to CRCP performance in the statewide condition survey. A further objective was to determine the causes of distress in this type of pavement. The field investigation, designed on the basis of the results of the statewide condition survey, was completed in the summer of 1973. This phase of the study was confined to the Interstate System in Indiana and included only 9 in. (23 cm) thick pavements.

The third phase of the study, currently in progress, consists of a laboratory testing program. This involves standard laboratory tests on samples of all pavement components obtained in the second phase of the work.

The design of the field investigation and the laboratory testing program necessitated that the data analysis be done in two parts. This paper reports the first part of the data analysis. In addition, the design of the field study, the test procedures used in the field investigation and the laboratory testing programs are described. Some of the results, relating to the effect of in-place concrete properties on CRC pavement distress, are presented.

DESIGN OF FIELD STUDY

The field investigation was designed to include only the CRC pavements that are part of the Interstate Highway System in Indiana. As a result only 9 in. (23 cm) thick pavements were evaluated. The test sections were located on Interstate highways I-65, I-69, I-70, I-90 and I-465.

The design of the field investigation was formulated on the results of the 1972 statewide condition survey. The factors that were found to be statistically significant in the condition survey were used, as the stratification criterion for sampling the test sections for the field study. The stratification scheme consisted of the following factors

- 1) Method of paving (slip-formed; side-formed)
- 2) Method of steel placement (depressed steel; steel pre-set on chairs)
- 3) Type of steel reinforcement (wire fabric; bar mats; loose bars)
- 4) Type of subbase (gravel; slag; crushed stone; bituminous stabilized)

A total of 31 CRCP test sections were included in the field investigation.

Delineation of Test Sections

The test sections used in the field investigation were delineated according to the following criteria:

1. The test section, 1000 ft (305 m) in length, was a tangent section under uniform grade conditions, i.e., completely under fill, cut or at grade conditions.

2. It was required that the test section lie within the internal portion of the continuous slab, substantially removed from construction or expansion joints.
3. The test section was located wholly within one physiographic unit, e.g., ground moraine, glacial terrace, flood plain, etc.
4. Wherever possible, a test section was located to include at least one location where significant distress as indicated by a breakup or a patch was observed.
5. The structural components of the pavement section were required to conform to a combination of levels of factors comprising the stratification scheme.
6. The 100 ft (305 m) test section was located within the randomly selected 5000 ft (1524 m) survey section used in the statewide conditions survey.

Only 15 sections were obtained that had at least one location where a breakup or a patch was observed. An additional section had one location with heavily spalled cracks indicative of an incipient failure condition. The rest of the sections showed no apparent indication of distress. In a few instances the 1000 ft (305 m) test sections could not be located within the 5000 ft (1524 m) randomly selected survey sections used in the statewide condition survey, because some elements of the controlling criteria could not be satisfied. In such cases, a new randomly sampled 1000 ft (305 m) test section was used.

Collection of Field Data

Figure 1 shows the layout of a typical test section. The data collected at a test section is also indicated. The first step in the data collection procedure was to divide the test section into 10 segments of 100 ft (30 m) length each. Within each test section, two test locations were selected. These test locations corresponded to failed and good pavement conditions respectively. A failed test location was defined as one showing distress as indicated by a breakup or a patch while a good test location was defined as one showing no apparent distress. In test sections showing no indication of a failed condition, also two test locations were used, one corresponding to an area with a normal and uniform crack pattern while the other to an area with an irregular and intersecting crack pattern. These crack patterns were evaluated subjectively by visual examination. In certain sections without any failures, only one test location was used because of limitation of time.

At a test location tests were conducted at two points. One test point, located at the pavement-shoulder interface, was designated as the shoulder position. The other test point was the hole through the pavement from which a concrete core had been extracted. This was designated as the core-hole position.

Since all tests at a test section had to be completed before proceeding to another test section a restriction on randomization was introduced. However, the order of conducting tests at various test locations and test points within a test

section was randomized.

A series of tests performed at a test section consisted of the following tests and measurements:

1. Deflection Measurements: These measurements were made by means of a Dynaflect (6). At the center of each 100 ft (30 m) segment two deflection measurements were taken; one at a crack position, while the other at the mid-span position between two transverse cracks. These measurements were taken along the center line of the traffic lane.

A second series of deflection measurements were made at the test locations. These measurements were taken across the traffic lane at 1.0 ft (0.30 m), 3.5 ft (1.07 m) and 6.0 ft (1.83 m) from the outside pavement edge, approximately corresponding to the outside edge, right wheel path and lane center line positions respectively. These transverse deflections were determined at both a crack position and an adjacent mid-span position between two transverse cracks.

Some additional deflection measurements were made at close intervals (about 2.0 ft or 0.61 m) at a failed location along the center line of the traffic lane in order to delineate the extent of the failure.

Hence, at a given test section, a minimum of 20 Dynaflect readings in the longitudinal direction and 12 Dynaflect readings in the transverse direction were recorded.

2. Crack Width Measurements: Crack widths were measured by means of a 50X, direct measuring pocket microscope. The points where crack width measurements were made corresponded to the positions along a crack where deflections were evaluated. This resulted in three crack width measurements at each test location or a total of six crack width measurements at each test section.
3. Crack Interval Measurements: Segments 50 ft (15 m) in length were staked out on either side of a test location. Then crack spacing was evaluated along the pavement edge over a 100 ft (30 m) section centered on a test location. That is, crack intervals were measured over a 100 ft (30 m) length of pavement at each test location within a test section. In addition, crack intersection points were counted over the 100 ft section at each test location.
4. Subgrade and Subbase Evaluation: In-place CBR tests on subbase and subgrade were made by means of the High Load Penetrometer (4) and the Dynamic Cone Penetrometer (7), respectively. These tests were performed at both core-hole and shoulder positions at each of the two test locations. The tests at the pavement-shoulder interface were replicated three times while the tests through the core-hole were done only once. In all eight penetrometer tests, four each on subbase and subgrade were performed at each test location or 16 penetrometer tests were performed on each test section.

In-place nuclear density and water content determinations were made on the subbase and subgrade. As a check, standard water content tests were also made on subbase and subgrade materials. In addition, subgrade density was measured by means of a thin-walled tube sampler to serve as a check on nuclear density readings. These tests were made at the shoulder position.

At the completion of a series of tests on the subbase and subgrade, material was sampled from under the pavement at the pavement-shoulder interface for laboratory testing.

5. Concrete Cores: Concrete cores were obtained from the two test locations within each test section. These cores were taken from the traffic lane, close to the point from where the subgrade and subbase material were sampled.

A description of equipment used in performing some of the field tests is given in Appendix 1.

LABORATORY TESTING PROGRAM

The laboratory testing program was planned in two parts:

1. Tests on Concrete Cores: Concrete cores obtained from the field were subjected to the following tests
  - a) Specific gravity and absorption tests
  - b) Pulse velocity measurements
  - c) Bulk density measurements

Next the cores were cut and segments from top and

bottom of the cores (without any steel) were tested for specific gravity, water absorption, pulse velocity, bulk density and splitting tensile strength.

2. Tests on Subbase and Subgrade Materials: The series of tests on subgrade soils and granular subbase materials include standard classification tests, compaction tests and some permeability tests (only on subbase aggregate mixtures). The stability characteristics and properties of bituminous stabilized subbase material would be evaluated by suitable laboratory tests.

The laboratory tests were performed according to ASTM Standards (1,2) where applicable. The concrete cores, subbase and subgrade materials were tested in a random order.

ANALYSIS AND RESULTS

The characteristics of the design of the field study suggested the following approach to data analysis:

1. Comparison of properties of failed test locations with good test locations within test sections, showing significant distress.
2. Comparisons of test sections showing distress, as indicated by a breakup or a patch, with test sections in good condition and showing no apparent distress.

This paper reports the findings obtained from the first part of the data analysis, that is, a comparison of structural characteristics of failed locations with good locations, within test sections showing significant distress.

The statistical analysis was conducted within the framework of a fixed-effect randomized complete block design (3). The main reason for using a randomized complete block design (RCBD) was to remove a source of variation, due to the effect of blocks, from the error term. The test sections of the field study design correspond to the blocks of the RCBD. The use of blocks, or test sections in the field study, imposed a restriction on randomization. As a result of this restriction the effect of test sections on various evaluated variables could not be tested for significance:

The dependent variables evaluated in this analysis are:

1. Subgrade CBR
2. Subbase CBR
3. Concrete properties - bulk unit weight, dynamic modulus of elasticity and splitting tensile strength
4. Dynamic pavement deflection
5. Crack width
6. Crack spacing
7. Crack intersections

Subgrade and Subbase CBR

Estimates of subgrade and subbase in-place California Bearing Ratio (CBR) values were obtained from the cone penetrometer test data. Subgrade CBR values apply to the top 4 in. (10 cm) of the subgrade layer. Subbase CBR data pertain to granular subbases. Hence, the results of this analysis are valid only for gravel, slag and crushed stone subbases.

The following analysis of variance model was used to analyze the subbase and subgrade CBR data:

$$Y_{ijk} = \mu + S_i + \delta(i) + C_j + SC_{ij} + P_k + SP_{ik} + CP_{jk} + \epsilon(ijk) \quad (1)$$

$$i = 1, 2, \dots, 14$$

$$j = 1, 2$$

$$k = 1, 2$$

where

Y_{ijk} = Estimated in-place CBR obtained at the k^{th} test point in j^{th} condition in the i^{th} test section (percent).

μ = Overall mean effect.

S_i = Effect of the i^{th} test section.

$\delta(i)$ = Restriction Error, random; NID $(0, \sigma^2)$
Completely confounded with the effect of the i^{th} test section, caused by running all tests at the i^{th} test section before proceeding to the next test section.

C_j = Effect of the j^{th} pavement condition (good location compared with failed location).

SC_{ij} = Interaction Effect of the i^{th} test section with the j^{th} pavement condition.

P_k = Effect of the k^{th} test point (shoulder vs. core-hole).

SP_{ik} = Interaction effect of the i^{th} test section with the k^{th} test point.

CP_{jk} = Interaction effect of the j^{th} pavement condition with the k^{th} test point.

$\epsilon_{(ijk)}$ = Error, NID $(0, \sigma^2)$. This term is estimated in the analysis of variance, from the interaction source assuming the interaction of the i^{th} test section, the j^{th} condition and the k^{th} test point is zero.

The results of the analysis of variance of subgrade and subbase CBR data are summarized in Tables 1 and 2, respectively. These results indicate:

1. There was a significant difference in subbase CBR between good and failed locations within a test section, while the difference in subgrade CBR values at these locations was not significant.
2. Subbase CBR values were significantly influenced by the position at which the test was performed. Tests performed at the core-hole gave higher CBR values than tests performed at the shoulder.
3. Relative to both subgrade and subbase CBR, the interaction effect between the characteristics of the test section and the pavement condition was significant.
4. The subbase CBR is significantly affected by the interaction between pavement condition and the position at which the test was performed.

The variation of subgrade and subbase CBR under various combinations of pavement condition and test points is shown in Table 3. The average values of subgrade and subbase CBR

tabulated in Table 3 were obtained from 14 test sections showing significant distress. As expected, subbase CBR values obtained at good locations were higher than the values tested at failed locations. The higher subbase CBR values obtained at core-hole points may be attributed to the confining effect of the pavement slab. In general, subbase CBR values were low pointing to the possible instability of this pavement layer in the pavement structure. In contrast, subgrade CBR values were about the same irrespective of pavement condition or location of test points.

Concrete Properties

The concrete properties evaluated in this analysis are bulk unit weight, dynamic modulus of elasticity and splitting tensile strength.

Bulk unit weight was calculated as follows:

$$\text{Bulk Unit Weight} = \frac{\text{Weight of sample at ambient moisture content}}{\text{Bulk volume of sample}} \quad (2)$$

Dynamic modulus of elasticity, E_c' , was estimated from pulse velocity determinations by the following relationship:

$$E_c' = (\text{Pulse Velocity})^2 \times \text{Density} \quad (3)$$

This estimated value deviates from the exact definitions of dynamic modulus of elasticity, E_c , which is given by:

$$E_c = (\text{Pulse Velocity})^2 \times \text{Density} \times \frac{(1+\mu)(1-2\mu)}{1+\mu} \quad (4)$$

where

μ = Poisson's ratio of concrete.

In this study, Poisson's ratio of concrete was not determined.

Concrete test data were analyzed by using the following analysis of variance model:

$$Y_{ijk} = \mu + S_i + \delta(i) + C_j + SC_{ij} + T_k + ST_{ik} + CT_{jk} + \epsilon(ijk) \quad (5)$$

$$i = 1, 2 \dots 15$$

$$j = 1, 2$$

$$k = 1, 2$$

where

Y_{ijk} = Concrete test value (bulk unit weight in lb/ft³; dynamic modulus of elasticity in psi; splitting tensile strength in psi.) obtained from testing the k^{th} segment of concrete core taken from a test location in j^{th} condition within the i^{th} test section.

μ = Overall mean effect.

S_i = Effect of the i^{th} test section.

$\delta(i)$ = Restriction error, random; NID $(0, \sigma^2)$.

C_j = Effect of the j^{th} pavement condition.

T_k = Effect of the k^{th} segment of concrete core.

$\epsilon(ijk)$ = Error, NID $(0, \sigma^2)$.

The other terms denote the two-factor interactions between the factors S_i , C_j and T_k . The pavement condition factor, C_j , implies a comparison of a good location with a failed location. Factor T_k refers to a comparison of the properties of the concrete core segment above the steel reinforcement with the segment below the steel.

Tables 4, 5 and 6 summarize the results of analysis of variance (ANOVA) of bulk unit weight, dynamic modulus of elasticity and splitting tensile strength data, respectively. An examination of these ANOVA results shows that:

1. A significant difference in bulk unit weight and dynamic modulus of elasticity of concrete was indicated between good and failed locations within a test section. The difference in splitting tensile strength at these locations was not significant.
2. No significant difference in bulk unit weight, modulus of elasticity or splitting tensile strength of concrete was detected between core segments above and below the steel reinforcement. In other words, properties of pavement concrete above and below the steel reinforcement were relatively uniform.
3. As in the case of subbase and subgrade strength characteristics, the interaction effect between the characteristics of the test sections (blocking effect) and the pavement condition was significant with respect to bulk density and modulus of elasticity of concrete.

These results were obtained from the analysis of concrete cores obtained from 15 test sections showing significant distress as evidenced by a breakup or a patch. Average values of bulk unit weight, dynamic modulus of elasticity and splitting tensile strength corresponding to failed and good pavement conditions are shown in Table 7. The difference in average

bulk unit weight of concrete shown in Table 7 does not show practical significance. However the statistical significance indicated in the analysis establishes that bulk unit weight of concrete at good locations was relatively higher than at failed locations.

From results presented in Tables 4, 5 and 7 it may be concluded that pavement failures, at sections showing distress, were associated with concrete having a relatively low bulk density and modulus of elasticity.

Table 8 shows the difference in concrete characteristics of core segments from above and below the steel reinforcement. These data indicate that there was no significant difference in the uniformity of concrete above and below the steel reinforcement relative to bulk density, tensile strength and modulus of elasticity.

Dynamic Pavement Deflections

Pavement deflections were measured, in mils (0.001 in.) by means of a Dynaflect, at specified intervals across the outside lane (traffic lane) at two test locations within a test section. The two test locations corresponded to failed and good conditions respectively. Only 12 out of the 15 test sections were used in the evaluation of pavement deflection characteristics, because of lack of complete data for two sections and exceedingly high deflections for the remaining sections, which had a bituminous stabilized subbase.

The following analysis of variance model was used for the analysis of deflection data:

$$\begin{aligned}
 Y_{ijkl} = & \mu + S_i + \delta(i) + C_j + SC_{ij} + L_k + SL_{ik} + \\
 & CL_{jk} + SCL_{ijk} + M_\ell + SM_{i\ell} + CM_{j\ell} + SCM_{ij\ell} + \\
 & LM_{k\ell} + SLM_{ik\ell} + CLM_{jk\ell} + \epsilon(ijkl)
 \end{aligned} \quad (6)$$

$$i = 1, 2, \dots, 12$$

$$j = 1, 2$$

$$k = 1, 2, 3$$

$$\ell = 1, 2$$

where

Y_{ijkl} = Deflection measurement in milli-inches made at the ℓ^{th} test position and the k^{th} transverse position on the outside lane, at a test location in j^{th} condition within the i^{th} test section.

μ = Overall mean effect.

S_i = Effect of the i^{th} test section.

$\delta(i)$ = Restriction error, random; NID $(0, \delta^2)$.

C_j = Effect of the j^{th} pavement condition (good vs failed)

L_k = Effect of the k^{th} transverse position.

M_ℓ = Effect of the ℓ^{th} test position (crack position vs mid-span position).

$\epsilon(ijkl)$ = Error, NID $(0, \delta^2)$.

The other terms denote the interaction effects among the factors S_i , C_j , L_k and M_ℓ . The factor L_k refers to transverse deflection measurements, made across the outside lane at 1.0 ft (0.30 m), 3.5 ft (1.01 m) and 6.0 ft (1.83 m) from the outside pavement edge. Essentially, this factor evaluates the variation of pavement deflection with distance from the pavement edge. The factor M_ℓ compares deflections obtained at a crack position with those measured at an adjacent mid-span position between two transverse cracks.

Table 9 gives a summary of the results of analysis of variance performed on deflection data. From these results, it may be concluded that:

1. There was a significant difference in pavement deflection between good and failed locations within a test section.
2. The difference in pavement deflection at various points across the outside (traffic) lane was significant.
3. There was no significant difference between deflections at crack positions as compared with deflections at mid-span positions.
4. The interaction between the test section characteristics and condition of test locations (good vs failed) had a significant effect on pavement deflection.
5. The following interaction effects also showed significant in the analysis of pavement deflections:

- a) interaction between test section characteristics and test positions (crack vs mid-span).
- b) interaction between the pavement condition (good vs failed), and distance from the outside edge of the pavement (transverse position factor).
- c) three-factor interaction among test section characteristics, pavement condition and test positions.

Figure 2 and Table 10 further explain some of the results of the analysis of variance. It would be noted in Table 10 that:

1. Average pavement deflection at good locations was about 58 percent of average deflection at failed locations.
2. Deflections decreased in magnitude with increasing distance from the outside pavement edge. On the average, deflection at the lane center line was 68 percent of that at 1.0 ft (0.3 m) from the outside edge.
3. Average deflection (0.87 milli-inches) at crack positions was not significantly different from the average deflection (0.93 milli-inches) at mid-span positions between transverse cracks. This could be attributed to the time of the year (June), when the study was conducted. Because of higher temperatures, the cracks are held close together, assuring greater

granular interlock and good load transfer. Figure 2 graphically illustrates some of the trends indicated by the results of the analysis of variance. From the standpoint of pavement condition, deflections close to the pavement edge were more critical than deflections at the interior of the traffic lane. The relative difference, in average pavement deflection, between good and failed locations was greater near the pavement edge than at more interior locations.

Crack Width

Measurements of crack width were made by means of a direct measuring pocket microscope with a 50X magnification. The microscope reticle was calibrated in increments of 0.001 in. A total of 16 test sections were included in this analysis. The additional test section was the one, having a test location showing heavily spalled cracks (incipient failure condition).

The ANOVA model used for the analysis of crack width measurements is as follows:

$$Y_{ijk} = \mu + S_i + \delta(i) + C_j + SC_{ij} + L_k + SL_{ik} + CL_{jk} + \epsilon(ijk) \quad (7)$$

$$i = 1, 2 \dots 16$$

$$j = 1, 2$$

$$k = 1, 2, 3$$

where

$$Y_{ijk} = \text{Crack width measurement in inch units made at the } k^{\text{th}} \text{ transverse position on the outside}$$

lane, at a test location in j^{th} condition within the i^{th} test section.

$$\epsilon_{(ijk)} = \text{Error, NID } (0, \sigma^2).$$

All other terms are as defined in Equation 6.

The results of the ANOVA are shown in Table 11 from which it may be concluded that:

1. A significant difference in crack width was indicated between good and failed locations within test sections.
2. The variation of crack width across the outside lane, between the outside edge and the lane center line, was not significant.
3. The interaction between the characteristics of test sections and pavement condition had a significant influence on crack width.

Further analysis of data showed that the mean crack width at good locations was 0.013 in. (0.33 mm) while at failed locations, the mean crack width was .032 in (0.81 mm). Some of the other relationships resulting from the analysis of variance are shown in Figure 3. The plotted data are average crack widths obtained at 16 test sections. It is indicated that crack widths at failed locations were consistently greater than crack widths at good locations irrespective of the transverse position across the outside lane.

Crack Spacing

Crack spacing was measured along the pavement edge over a distance of 100 ft (30 m) at each test location within a

test section. The measures of crack spacing used in the analysis are:

1. Mean crack interval, $\bar{x}_c$ (ft)
2. Standard deviation of crack interval, S_c (ft)
3. Coefficient of variance of crack interval, $\bar{CV}_c$ (percent).

$$\text{where, } \bar{CV}_c = \frac{S_c}{\bar{x}_c} \times 100 \quad (8)$$

These variables were analyzed by ANOVA procedures by means of the following model:

$$Y_{ij} = \mu + S_i + \delta(i) + C_j + \epsilon_{(ij)} \quad (9)$$

$$i = 1, 2 \dots 15$$

$$j = 1, 2$$

where

Y_{ij} = A measure of crack spacing, e.g., $\bar{x}_c$, S_c , $\bar{CV}_c$ obtained at a test location in j^{th} condition within i^{th} test section.

$\epsilon_{(ij)}$ = Error, NID $(0, \sigma^2)$.

and other terms are as defined in Equation 1.

For sake of brevity, ANOVA results are not reproduced here. However, these results indicated that:

1. There was no significant difference in mean crack interval, $\bar{x}_c$ between good and failed locations within a test section.
2. The difference in standard deviation, S_c and coefficient of variance, $\bar{CV}_c$ of crack interval, between good and failed locations was significant.

Table 12 shows the relationship between pavement condition and certain statistical measures of crack spacing. It would be observed that mean crack interval was not a reliable indicator of pavement condition. A measure of uniformity of crack spacing would be more appropriate from the viewpoint of ascertaining pavement conditions. The results shown in Table 12 indicate that pavement sections in good condition had a more uniform or less dispersed crack spacing than failed sections, the measure of uniformity of crack spacing being either the standard deviation or the coefficient of variance of crack spacing.

Crack Intersections

It was observed in previous condition surveys that good pavement condition was associated with cracks that had a parallel transverse trend with relatively few or no intersections. Consequently in the field study, this aspect was investigated in more detail. At each test location in a test section the number of crack intersections observed per 100 ft (30 m) length of pavement were counted.

The ANOVA model used for analyzing crack intersections data was obtained as:

$$Y_{ij} = \mu + S_i + \delta_{(i)} + C_j + \epsilon_{(ij)} \quad (10)$$

$$i = 1, 2 \dots 15$$

$$j = 1, 2$$

where ,

Y_{ij} = Number of crack intersections/100 ft length of pavement observed at the test location in j^{th} condition within the i^{th} test section.

$\epsilon_{(ij)}$ = Error, NID $(0, \sigma^2)$.

and other terms are as defined in Equation 1.

The results of analysis of variance are given in Table 13. It would be observed that there was a significant difference in number of crack intersections per 100 ft (30 m) length of pavement, between good and failed locations within a test section. Average values of this variable at good and failed locations are indicated in Table 12. These results show that random and irregular crack patterns are indicative of poor pavement condition whereas uniform, evenly spaced crack patterns with relatively few intersecting cracks are representative of good pavement condition.

CONCLUSIONS

The results reported in this paper are based on a comparison of properties of failed test locations with good test locations, within test sections showing significant distress, as indicated by a breakup or a patch. These results were derived from a statistical analysis of a portion of pavement condition data obtained from a field investigation of CRC pavements in Indiana. As such, the conclusions based on these results have a limited inference space and do not apply to a comparison of test sections showing adequate performance with those indicating significant distress as evidenced by a breakup or a patch.

The following conclusions pertain to an evaluation of parameters, relating to the performance of CRC pavements in Indiana.

1. Subgrade CBR. No significant difference in subgrade CBR was indicated between good and failed locations.
2. Subbase CBR. Subbase CBR values obtained at good locations were higher than the values obtained at failed locations. Higher CBR values were obtained at the core-hole than at the shoulder. The higher values at the core-hole positions could be due to the confining effect of the pavement slab.
3. Concrete Properties. Pavement failures were associated with concrete having relatively low bulk density and modulus of elasticity. No significant difference in splitting tensile strength was indicated between good and failed locations. Also, there was no significant difference in the uniformity of concrete above and below the steel reinforcement with respect to bulk density, tensile strength and modulus of elasticity.
4. Dynamic Pavement Deflection. Average pavement deflection at good locations was about 58 percent of average deflection at failed locations. Higher deflections were observed at the pavement edge than at the interior of the outside lane. Deflections decreased in magnitude with increasing distance from the pavement edge. Deflections at crack positions were not significantly different from deflections at

mid-span positions between transverse cracks. From the standpoint of pavement condition, deflections close to the pavement edge were more critical than deflections at the interior of the outside lane.

5. Crack Width. Mean crack width at good locations was 0.013 in (0.33 mm) while the mean crack width at failed locations was 0.032 in (0.81 mm).
6. Crack Spacing. No significant difference in mean crack interval was indicated between good and failed locations. However, a measure of variance of crack interval, indicative of uniformity of crack spacing, yielded more promising results. It was shown that the standard deviation and coefficient of variance of crack interval at good locations were significantly different from those at failed locations.
7. Crack Intersections. A large number of intersecting cracks were indicated at failed locations than at good locations thereby indicating that good pavement condition is associated with uniform, evenly spaced transverse crack patterns.

APPENDIX 1. DESCRIPTION OF EQUIPMENT USED IN THE CRCP FIELD STUDY

1. HIGH LOAD PENETROMETER. This is a static soil strength tester. It consists of a two inch diameter cone point mounted at the rod end of a hydraulic cylinder. The hydraulic cylinder or jack is connected by a hose to a hand pump which provides the hydraulic pressure to extend the cylinder. This arrangement provides a large penetration force on the test probe. The test probe is sized so that the effect of gravel, in the range of sizes commonly found, can be included as part of the measured soil strength. The penetrometer is calibrated in terms of California Bearing Ratio (CBR). The CBR of soil can be determined as a function of the soil failure pressure under the influence of the cone point. The relationship of CBR and this failure pressure is expressed as:

$$CBR = \frac{P_s}{25} - \frac{P_s^2}{416670} \quad (A1)$$

where

CBR = California Bearing Ratio (percent)

P_s = Soil failure pressure in psi exerted on the projected area of the 2 in. diameter cone point (3.14 in.^2).

Any friction and shear forces at the side of the cone point are included in the total force.

The High Load Penetrometer was used for subbase evaluation in the field study of CRC pavements.

2. DYNAMIC CONE PENETROMETER. The Dynamic Cone Penetrometer (DCP) is a modified version of the penetrometer used by the Country Roads Board, Australia. It consists basically of a 10 kg hammer sliding on a 16 mm rod dropping through a distance of 460 mm and striking an anvil at the lower end of that rod, on the end of which is a hardened steel cone, 20 mm in diameter. The penetrometer is driven by blows of the drop hammer and the penetration per blow in mm is measured on graduations on the upper rod and recorded. The CBR value of the in-situ soil can be obtained from penetration versus CBR calibration. The bearing value of soils in the range of CBR 1 to 50 can be rapidly determined by the use of the DCP. It should, however, be noted that the DCP is not suitable for use in granular soils, coarse sand probably being the limit of usability. In the field investigation of CRC pavements, DCP was used for subgrade evaluation.
3. DIRECT MEASURING MICROSCOPE. Crack widths were measured by means of a 50X, pocket size direct measuring microscope. This is a handy microscope with a precision glass reticle having a 0.1 in. scale calibrated in increments of 0.001 in. Estimates of up to 0.0005 in. can be made by this device.
4. DYNAFLECT. This is an electro-mechanical system for measuring the dynamic deflection of a surface or structure caused by an oscillatory load. Deflection measurements are independent of a fixed surface reference. The deflection readings obtained by this system range from 30 milli-inches (.03 in) to ten micro-inches (.00001 in).

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Table 1. Summary of Analysis of Variance - Subgrade CBR (Percent)

S = Significant   NS = Non-Significant

Source of Variation	DF	Sums of Squares	Mean Square	F	F .05	Significance ($\alpha = .05$)
Test Sections, S_i	13	3504.23	269.56	*	2.58	--
Restriction Error, $\delta (i)$	0	--	--	--	--	--
<u>Main Effects</u>						
Condition, C_j	1	13.02	13.02	0.39	4.67	NS
Test Points, P_k	1	58.02	58.02	1.73	4.67	NS
<u>Interaction Effects</u>						
SC_{ij}	13	1174.23	90.33	2.69	2.58	S
SP_{ik}	13	706.23	54.33	1.62	2.58	NS
CP_{jk}	1	2.16	2.16	0.06	4.67	NS
Error, $\epsilon (ijk)$	13	437.09	33.62			
Total	55	5894.98				

* None available because of restriction on randomization due to the blocking effect of test sections.

Table 2. Summary of Analysis of Variance - Subbase CBR (Percent)

S = Significant   NS = Non-Significant

Source of Variation	DF	Sums of Squares	Mean Square	F	F .05	F .10	Significance
Test Sections, S_i	13	10838.81	833.75	--	2.58	2.10	--
Restriction Error, $\hat{q}_i$	0	--	--	--	--	--	--
<u>Main Effects</u>							
Condition, C_j	1	196.88	196.88	4.64	4.67	3.14	S ($\alpha = .10$)
Test Points, P_k	1	986.16	986.16	23.26	4.67	3.14	S ($\alpha = .05$)
<u>Interaction Effects</u>							
SC_{ij}	13	2824.38	217.26	5.12	2.58	2.10	S ($\alpha = .05$)
SP_{ik}	13	1420.09	109.24	2.58	2.58	2.10	S ($\alpha = .10$)
CP_{jk}	1	123.02	123.02	2.90	4.67	3.14	NS
Error, ϵ_{ijk}	13	551.23	42.40				
Total	55	16940.55					

Table 3. Variation of Subbase and Subgrade CBR With Pavement Condition and Location of Test Points*

	Pavement Condition			
	Good		Failed	
	Shoulder	Core-Hole	Shoulder	Core-Hole
Average Subbase CBR (Percent)	39.1	44.5	32.4	43.7
Average Subgrade CBR (Percent)	14.6	16.3	13.3	15.7

*Average of CBR data obtained from 14 test sections.

Table 4. Summary of Analysis of Variance - Bulk Unit Weight of Concrete (lb/ft^3)

S = Significant   NS = Non-Significant

Source of Variation	DF	Sums of Squares	Mean Square	F	F .05	Significance ($\alpha = .05$)
Test Sections, S_i	14	465.60	33.26	--	2.42	--
Restriction Error, $\delta_{(i)}$	0	--	--	--	--	--
<u>Main Effects</u>						
Condition, C_j	1	12.24	12.24	5.54	4.60	S
Core Segments, T_k	1	4.87	4.87	2.21	4.60	NS
<u>Interaction Effects</u>						
SC_{ij}	14	101.77	7.27	3.29	2.42	S
ST_{ik}	14	71.61	5.12	2.32	2.42	NS
CT_{jk}	1	1.57	1.57	0.71	4.60	NS
Error, $\epsilon_{(ijk)}$	14	30.94	2.21			
Total	59	688.61				

Table 5. Summary of Analysis of Variance - Dynamic Modulus of Elasticity (10^6 psi)

S = Significant   NS = Non-Significant

Source of Variation	DF	Sums of Squares	Mean Square	F	F .05	Significance ($\alpha = .05$)
Test Sections, S_i	14	6.031	0.431	--	2.42	--
Restriction Error, $\delta_{(i)}$	0	--	--	--	--	--
<u>Main Effects</u>						
Condition, C_j	1	1.362	1.362	24.39	4.60	S
Core Segments, T_k	1	0.001	0.001	0.02	4.60	NS
<u>Interaction Effects</u>						
SC_{ij}	14	4.599	0.329	5.88	2.42	S
ST_{ik}	14	1.008	0.720	1.29	2.42	NS
CT_{jk}	1	0.088	0.088	1.58	4.60	NS
Error, $\epsilon_{(ijk)}$	14	0.782	0.056			
Total	59	13.872				

Table 6. Summary of Analysis of Variance - Splitting Tensile Strength (psi)

S = Significant   NS = Non-Significant

Source of Variation	DF	Sums of Squares	Mean Square	F	F .05	Significance ($\alpha = .05$)
Test Sections, S_i	14	191252.46	13660.9	--	2.49	
Restriction Error, $\delta_{(i)}$	0	--	--	--	--	
<u>Main Effects</u>						
Condition, C_j	1	7526.4	7526.4	2.33	4.60	NS
Core Segments, T_k	1	1306.7	1306.7	0.41	4.60	NS
<u>Interaction Effects</u>						
SC_{ij}	14	61738.1	4409.9	1.36	2.49	NS
ST_{ik}	14	58899.8	4207.1	1.30	2.49	NS
CT_{jk}	1	3969.1	3969.1	1.23	4.60	NS
Error, $\epsilon_{(ijk)}$	14	45312.4	3236.6			
Total	59	370004.9				

Table 7. Relationship Between Pavement Condition and Concrete Properties *

Concrete Properties	Pavement Condition	
	Good	Failed
Average Bulk Density, γ_c lb/ft ³ (kg/m ³)	141.3 (2264)	140.3 (2248)
Average Dynamic Modulus of Elasticity, E_c 10 ⁶ psi (10 ⁶ kgf/cm ²)	5.01 (0.352)	4.71 (0.331)
Average Splitting Tensile Strength, f_t psi (kgf/cm ²)	525.8 (36.97)	503.3 (35.39)

*Average of Test Data from 15 test sections.

Table 8. Concrete Characteristics Above and Below The Steel Reinforcement*

Concrete Properties	Above the Steel Reinforcement	Below the Steel Reinforcement
Average Bulk Density, γ_c lb/ft ³ (kg/m ³)	141.6 (2268)	141.1 (2260)
Average Dynamic Modulus of Elasticity, E_c 10 ⁶ psi (10 ⁶ kg/cm ²)	4.85 (0.341)	4.86 (0.342)
Average Splitting Tensile Strength, f_t psi (kgf/cm ²)	509.9 (35.85)	519.2 (36.50)

*Average of test data from 15 test sections.

Table 9. Summary of Analysis of Variance - Dynamic Pavement Deflection (milli-inches)

S = Significant   NS = Non-Significant

Source of Variation	DF	Sums of Squares	Mean Square	F	F .05	Significance ($\alpha = .05$)
Test Sections, S_i	11	9.04	0.82	--	2.27	--
Restriction Error, $\delta_{(i)}$	0	--	--	--	--	--
<u>Main Effects</u>						
Condition, C_j	1	8.04	8.04	105.26	4.30	S
Transverse Positions, L_k	2	3.22	1.61	21.08	3.44	S
Test Positions, M_l	1	0.13	0.13	1.73	4.30	NS
<u>Interaction Effects</u>						
SC_{ij}	11	5.64	0.51	6.72	2.27	S
SL_{ik}	22	2.89	0.13	1.72	2.05	NS
SM_{il}	11	4.32	0.39	5.15	2.27	S
CL_{jk}	2	1.03	0.52	6.76	3.44	S
CM_{jl}	1	0.27	0.27	3.52	4.30	NS
LM_{kl}	2	0.01	0.005	0.07	3.44	NS
SCL_{ijk}	22	2.41	0.11	1.43	2.05	NS
SCM_{ijl}	11	4.22	0.38	5.02	2.27	S
SLM_{ikl}	22	1.91	0.09	1.13	2.05	NS
CLM_{jkl}	2	0.01	0.004	0.05	3.44	NS
Error, $\epsilon_{(ijkl)}$	22	1.68	0.076			
Total	143	44.82				

Table 10. Relationship Between Pavement Deflection and Evaluated Factors

Average Dynamic Deflection, 10^{-3} in. (10^{-3} cm) [†]					
Pavement Condition		Transverse Location			Test Position
Good	Failed	O.E.*	R.W.P.**	L.C.***	At Crack
0.66 (1.68)	1.14 (2.90)	1.10 (2.79)	0.85 (2.16)	0.75 (1.91)	0.87 (2.21)
					Between Cracks
					0.93 (2.36)

† Average of Deflection measurements at 12 test sections

* O.E. = Outside Edge, 1.0 ft (0.3 m) from pavement edge

** R.W.P. = Right Wheel Path, 3.5 ft (1.07 m) from pavement edge

***L.C. = Lane Center-Line, 6.0 ft (1.83 m) from pavement edge.

Table 11. Summary of Analysis of Variance - Crack Width (in.)

S = Significant   NS = Non-Significant

Source of Variation	DF	Sums of Squares	Mean Square	F	F .05	Significance ($\alpha = .05$)
Test Sections, S_i	15	0.1229	0.00082	--	2.01	--
Restriction Error, $\delta_{(i)}$	0	--	--	--	--	--
<u>Main Effects</u>						
Condition, C_j	1	0.00851	0.00851	44.79	4.17	S
Transverse Location, L_k	2	0.00063	0.00032	1.68	3.32	NS
<u>Interaction Effects</u>						
SC_{ij}	15	0.00698	0.00047	2.47	2.01	S
SL_{ik}	30	0.00366	0.00012	0.63	1.84	NS
CL_{jk}	2	0.00124	0.00062	3.26	3.32	NS
Error, $\epsilon_{(ijk)}$	30	0.00579	0.00019			
Total	95	0.03911				

Table 12. Relationship Between Statistical Measures of Crack Spacing,
Crack Intersections and Pavement Condition

Pavement Condition	Mean Crack Interval, $\bar{X}_c$		Standard Deviation of Crack Interval, S_c		Coefficient of Variation of Crack Interval, $\overline{CV}_c$		Average Number of Crack Intersections Per 100 ft. Length of Pavement
	Average ft.(m)	Range ft.(m)	Average ft.(m)	Range ft.(m)	Average (Percent)	Range (Percent)	
Good	5.5 (1.7)	2.5-9.0 (0.76-2.7)	2.45 (0.747)	1.91-4.11 (0.582-1.25)	48.3	28-71	2.1
Failed	4.7 (1.4)	2.4-9.1 (0.73-2.8)	2.96 (0.902)	1.78-3.12 (0.543-0.951)	65.5	45-88	4.9

Note: Average and Range Values Obtained from 15 Test Sections.

Table 13. Summary of Analysis of Variance - No. of Crack Intersections/100 Ft.
Length of Pavement

Source of Variation	S = Significant			NS = Non-Significant		
	DF	Sums of Squares	Mean Square	F	F .05	Significance ($\alpha = .05$)
Test Sections, S_i	14	146.00	10.43	--	2.49	--
Restriction Error, $\delta_{(i)}$	0	--	--	--	--	--
Condition, C_j	1	61.63	61.63	14.41	4.60	S
Error, $\epsilon_{(ij)}$	14	59.87	4.28			
Total	29	267.50				

- LEGEND :
- Δ = DYNAFLECT MEASUREMENTS AT CRACK AND MID-SPAN POSITIONS
  - w = CRACK WIDTH MEASUREMENTS AT ONE CRACK
  - = CONE PENETROMETER TEST FOR SUBBASE
  - x = DYNAMIC CONE PENETROMETER TEST FOR SUBGRADE
  - D = SUBGRADE AND SUBBASE DENSITY AND WATER CONTENT MEASUREMENTS
  - CI = 100ft. SECTION FOR CRACK INTERVAL MEASUREMENTS AND CRACK INTERSECTION COUNTS

FIG.1 TYPICAL LAYOUT OF TEST SECTIONS (NOT TO SCALE)

FIG. 2 EFFECT OF POSITION AND CONDITION ON PAVEMENT DEFLECTION (MEASUREMENTS MADE IN TRAFFIC LANE)

FIG. 3 RELATIONSHIP OF CRACK WIDTH WITH POSITION AND PAVEMENT CONDITION (MEASUREMENTS MADE IN TRAFFIC LANE).

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