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

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Evaluation of Several Maintenance Methods for Continuously Reinforced Concrete Pavement

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Research on continuously reinforced concrete pavement (CRCP) has been going on at Purdue University for the Indiana State Highway Commission since 1971. The primary objective of the overall research program has been to evaluate the performance of CRCP in Indiana and to make recommendations relative to design and construction techniques that might improve the performance of this type of pavement. Primary factors found to contribute to performance of CRCP in Indiana have been subbase type, method of construction, and traffic. The usual method of maintaining CRCP has been to patch failures by using reinforced concrete. This research has been done to evaluate other techniques for maintaining CRCP to determine the most cost-effective method. A test pavement on Interstate-65 south of Indianapolis was used. Maintenance methods investigated included normal concrete patching, bituminous patching, overlay by using asphalt concrete with and without prior undersealing and with and without installation of edge drains, undersealing by using asphalt only, drainage, and concrete shoulders. The results show that overlaying the pavements by using asphalt concrete completely stopped the progression of failures within the time frame of this research. Undersealing by using asphalt was also effective. The edge drains and concrete shoulders were not effective. The performance data were also analyzed in light of the cost of maintenance.

The use of continuously reinforced concrete pavement (CRCP) in Indiana dates back to 1938, when an experimental project was first built on US-40 near Stilesville, Indiana. The mileage of CRCP increased until the end of 1971, when 1120 km (696 miles) of equivalent two-lane CRCP were in service in the state.

In 1973, a continuing study of the performance of CRCP was initiated by the Joint Highway Research Project at Purdue University. The objective of the study was to evaluate performance and to recommend design and construction techniques.

HISTORY OF RESEARCH ON CRCP IN INDIANA

Primary emphasis was placed on construction of CRCP from 1967 to 1971. By the spring of 1972 it became apparent that distress was occurring on some of the pavements. Purdue University was first contacted in July 1972 regarding the problem; at that time, plans were made for a long-range research project.

The procedure below briefly summarizes the process of the research.

1. Detailed study of performance on Interstate-65, a major Interstate between Indianapolis and Chicago;
2. Statewide performance survey of all CRCP in Indiana;
3. Detailed study of selected pavements, including field measurements;
4. Laboratory evaluation of materials obtained in step 3;
5. Analysis of factors that influence performance;
6. Construction of test sections of several types of maintenance in the fall of 1975 on I-65 south of Indianapolis; and

7. Evaluation of test pavements through the spring of 1979.

Significant Factors That Influence Performance

After several months of research on CRCP in Indiana, it became apparent that several factors were showing up as statistically significant with respect to design and performance of this type of pavement in Indiana. These factors have been discussed in detail in several reports (1-4). The factors that influence the performance of CRCP include (a) subbase type, (b) type of steel fabrication, (c) interaction between steel placement methods and construction, and (d) traffic.

In the following discussion, some of the features that have been found to influence the performance of CRCP are mentioned. They are not necessarily discussed in the order of importance, since it is believed that there is no single factor that has influenced performance. Rather, it is a combination of factors that have set up a series of circumstances that have caused failures to occur.

Subbase Type

The early surveys showed that pumping was occurring on the pavements that showed the greatest amount of distress. Most of the pumping was occurring on the gravel subbases, whereas the crushed-stone and slag subbases were apparently showing good performance.

Table 1 describes the variation in California bearing ratio (CBR), permeability K, and degree of compaction with subbase type. Although no clear differences in the properties of the gravel subbases were evident between sections that had failures and those that showed no apparent distress, the gravel subbases were not sufficiently compacted and in addition had relatively low permeability and strength.

The results brought to light important differences among the properties of the different subbase types. Crushed-stone subbases were the most permeable, whereas slag subbases had the lowest permeability. The relatively poor water-transmission characteristic of the slag subbase was more than balanced by its high strength, as indicated by the CBR.

It is worthwhile to note that concrete pavement performance is also a function of the interaction between subbase permeability and strength (CBR). In Figure 1, the estimated field permeability values are plotted against field subbase CBR values measured at the interface between the shoulder and the slab. These values pertain to 46 test locations of the detailed field study. Data points for crushed-stone and slag subbases are shown by using subscripts. In addition, values obtained at failed test locations are differentiated from the values at good test locations. The data were grouped in nine categories that corresponded to three levels each of subbase CBR and permeability. For low subbase strength (CBR < 40 percent), mainly gravel subbases, the percentage of failed test locations decreased from 53 percent in the low-permeability group [K < 0.05 cm/s (142 ft/day)] to 25 percent in the high-permeability group [K > 0.5 cm/s (1418 ft/day)]. For medium subbase strength (40 percent < CBR < 80 percent), no failures were observed when permeability was greater than 0.5 cm/s. When subbase strength (CBR > 80 percent) was high (only slag and crushed-stone subbases), no failures were indicated irrespective of permeability.

It had been suggested in reports submitted to the Indiana State Highway Commission from the outset that pumping was a major contributor to pavement distress and that the gravel subbases have shown the poorest performance. The cold-mix bituminous stabilized bases showed fair to good performance. It was recommended that the Indiana State Highway Commission discontinue use of gravel subbases and that crushed-stone subbases (drained) or stabilized subbases be used. Data obtained suggest that slag subbases are showing good performance.

Percentage of Steel

A standard of 0.6 percent steel had been adopted by most states for use in CRCP up to the time of this research.

The temperature drop used by the Indiana State Highway Commission for the design of these pavements is 38°C (100°F). However, in the central and northern portions of Indiana, temperature changes from extreme highs during the construction season in midsummer to lows in the winter may be as great as 52°C (125°F) or more (4). According to computations of the effect of total temperature drop on required percentages of steel by using split-tensile-strength values from pavement cores, a temperature drop of 52°C requires more than 0.6 percent steel. This suggests that 0.6 percent steel was too low for conditions in Indiana.

Steel Placement

The survey data have shown that a major contributor to failures that occur on CRCP in Indiana can be associated with use of preset steel set on chairs. It was recommended that the state discontinue use of preset steel. Performance surveys suggested that bar mats showed the poorest performance.

Indiana pavement design has followed the standard recommendations of using a steel reduction factor of about 0.75. Various individuals in the United States have suggested that reduction in thickness should not be permitted. In spite of the lack of substantiating data, it is probable that the thickness design used on CRCP in Indiana was at best marginal.

Construction and Other Factors

The discussion so far has dealt primarily with factors that can be assumed to be in the category of design. The total research study, however, has dealt with other factors that might affect the performance of CRCP. Reference is made in particular to the report by Faiz and Yoder (3) that dealt with physical properties determined in the field on selected pavements during the summer of 1973.

Among the more-important significant factors was the effect of the bulk density of the concrete on performance of the pavements. Concrete at failed locations had lower bulk densities than did that at good locations. Likewise, the dynamic modulus of elasticity was found to be a contributing factor. On the other hand, the split-tensile-strength values were not found to be significantly different between failed and structurally sound locations.

The results of the study showed that there was little difference between the properties above and below the steel reinforcement.

The results of the field survey indicated that no significant difference existed in the mean crack interval between good and failed locations. It was shown that most of the failed sections were associated with intersecting cracks.

Summary of Field Performance Findings

The comments offered below pertain to research on

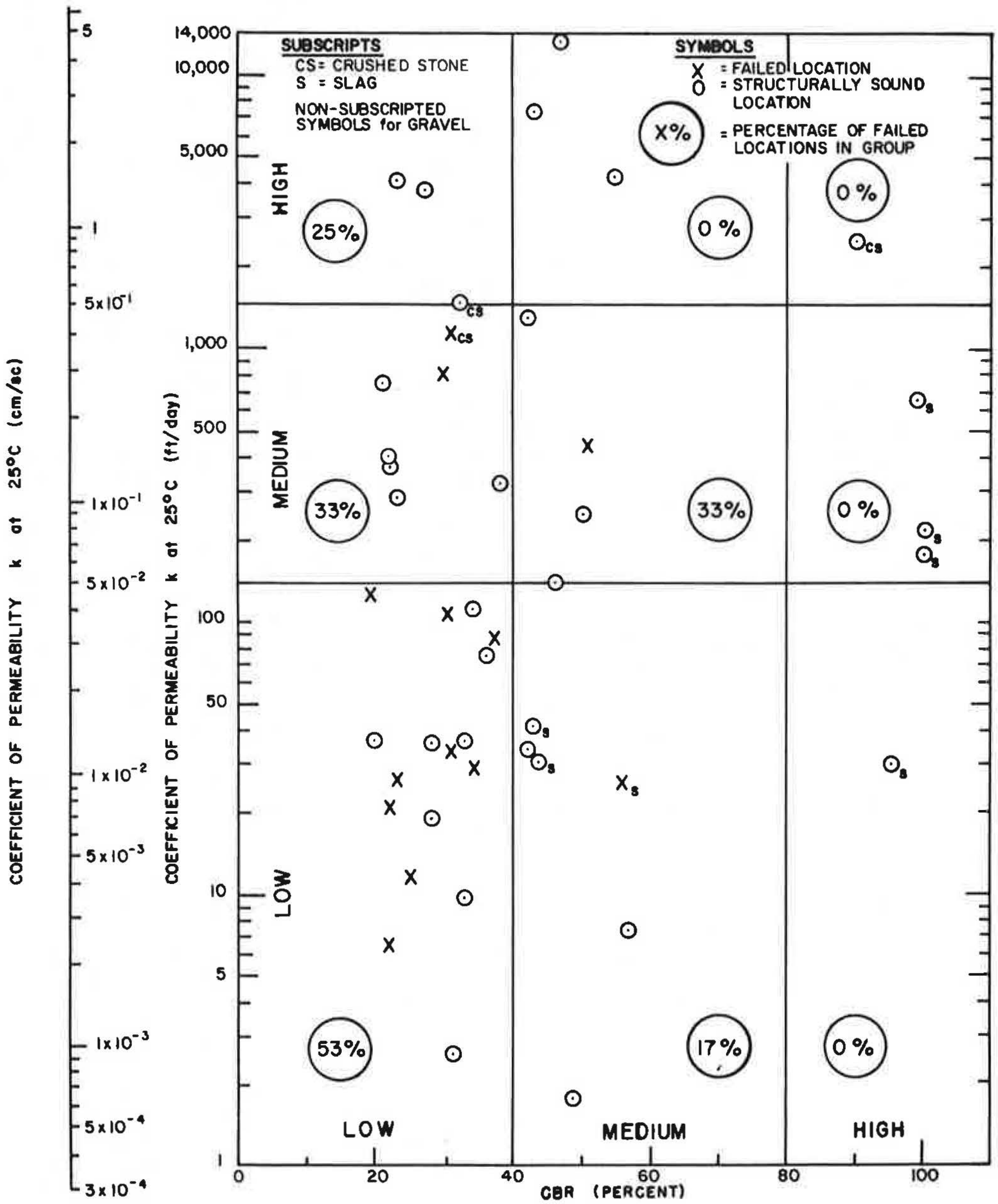
Table 1. Effect of subbase type on pavement condition.

Type of Subbase	Condition of Test Section	No. of Test Sections	Subbase CBR (%) Sampled at Pavement Edge	Subbase CBR (%) Sampled at Center Through Core Hole	Permeability K (cm/s)	Compaction (%)
Gravel	With failures	13	33.5	38.0	0.4[a]	91.5[a]
	Without failures	10	35.9	45.2	0.25[b]	93.2[b]
Slag	With failures	1	95.0	100.0	0.01	117.2
	Without failures	3	81.0	96.6	0.05	100.6
Crushed stone	With failures	1	32.0	41.0	0.7	89.9
	Without failures	1	90.0	59.0	1.5	85.3

Notes: Data are from structurally sound test locations.
1 cm/s = 2835 ft/day.

[a]Average of values from nine test sections. [b]Average of values from 10 test sections.

Figure 1. Effect of subbase strength and permeability on CRCP performance.



factors that have influenced performance of CRCP in Indiana.

In the detailed field evaluation, the comparison of test sections that had failures with sections that did not have failures relative to material properties and performance characteristics evaluated at structurally sound test locations resulted in a number of significant results. Similarly, the evaluation of sectionwide pavement characteristics also established some significant trends. These findings bring to light deficiencies inherent in the pavement structure that eventually lead to distress. The following is a summary of the significant results:

1. Subgrade properties: The only significant result in the analysis of subgrade properties was that subgrade soils at sections without failures were relatively more coarse grained and sandy than those soils at sections in which failures had occurred.

2. Subbase properties: This analysis clarified the reasons for the better performance of certain subbase types. Crushed-stone subbase at the section without failures was found to possess high strength (CBR, 90 percent) and excellent internal drainage [more than 0.7 cm/s (1985 ft/day)]. The failure at another section that had a crushed-stone subbase was a function of poor stability (very low CBR), which resulted from inadequate compaction. The good condition of pavements on slag subbases was due to the very high stability (CBR, more than 100 percent) of this subbase. At structurally sound locations, gravel subbases were found to have moderately high permeability but poor stability characteristics, probably a function of insufficient compaction.

3. Concrete properties: Sections that showed no failures were paved with concrete that had a higher degree of slump. The results of the data analysis further indicated that the modulus of elasticity of concrete had a significant bearing on pavement condition. Concrete cores obtained from sections that had no distress were tested to have an average dynamic modulus of elasticity of 4.32×10^9 kPa (6.15×10^6 psi), whereas cores obtained from good locations on sections that had failures had an average dynamic modulus of 3.49×10^9 kPa (4.97×10^6 psi).

4. Dynamic pavement deflection: Dynamic pavement deflections were shown to be a good indicator of pavement condition if used judiciously. Once the continuous slab breaks up into discrete segments, the usefulness of deflection measurements is impaired. As expected, at good test locations no difference in dynamic deflections was observed between sections that had failures and sections that did not have failures.

An evaluation of sectionwide deflection measurements taken 1.82 m (6.0 ft) from the pavement edge showed that for 228-mm (9-in) CRCP, dynamic deflections measured by Dynaflect less than 127×10^{-2} mm (0.5 mil) are indicators of good pavement condition. Deflections in the range of $1.54\text{-}2.28 \times 10^{-2}$ mm (0.6-0.9 mil) indicate a potential distress condition, whereas values above 2.28×10^{-2} mm are indicators of severe distress in areas where there is a high probability of pavement breakups.

5. Crack width: It was noted that cracks observed at test sections that had failures were significantly wider than those measured at good test sections, even though crack widths were measured only at structurally intact locations. The average crack width at good sections was 0.218 mm (8.6×10^{-2} in).

6. Crack spacing: No difference in either the mean crack spacing or the variance of crack intervals was observed between sections in the two categories. The variance of crack spacing at failed test locations was significantly higher than the variance at good locations. Frequent incidence of bifurcated cracks as well as closely spaced cracks that may intersect at a later date was observed to be associated with failures. Also, high incidence of very closely spaced cracks is indicative of incipient failure.

PURPOSE OF RESEARCH

It was the purpose of this research to design, construct, and evaluate several different types of maintenance for CRCP. Deflection, the amount of cracking, and the amount of breakups were used as significant indicators of performance and were the basis of selection of the experimental sections.

Previous research on CRCP in Indiana suggested that deflections measured by the Dynaflect that are greater than 2.3×10^{-2} mm indicate poor pavement condition. Hence, it follows that reduction of deflections is one method of reducing potential failure of the pavement. Poor drainage, often caused by the previously mentioned factors, was considered to be a major contributor to high deflection values. A third factor was found to be the use of bar mats and preset steel on chairs.

A section of I-65 south of Indianapolis was chosen for this project. This section extends south from the Greenwood exit of I-65 to the Whiteland exit and the test pavements included both the northbound and southbound lanes. This resulted in approximately 7.35 km (4.57 miles) in each direction or a total of about 14.71 km (9.14 miles) of test pavement.

This section was selected for study since it contains all the significant features identified as major contributors to poor performance of CRCP. It has a gravel subbase, and bar mats and chairs were used--three important criteria discussed earlier that indicate potential failure. The grade is relatively flat, soils are fine-grained glacial till, and drainage is generally poor. This pavement has shown, as predicted, very poor performance.

METHOD OF ESTABLISHING TEST SECTIONS

Initial Tests

Deflection measurements and a condition survey of the test pavement were made in the fall of 1974. Deflection readings were taken by using the Dynaflect at 7.6-m (25-ft) intervals over the study area. The condition survey was made by noting the size and location of breakups and the location and linear feet of closely spaced parallel cracks, intersecting cracks, and combined parallel and intersecting cracks. Figure 2 shows the criteria used for determining the linear feet of cracking. Linear feet of cracking (L, Figure 2) was the longitudinal distance observed for a specified type of crack.

Selection of Study Sections

By using the data derived from the above field observations, three factors were chosen as indicative of the overall condition of the pavement. These were (a) linear distance of cracks less than 762 mm (30 in) plus distance of intersecting cracks per 30.48-m (100-ft) section, (b) total area of patching or breakups per station, and (c) maximum deflection per 30.48-m section. By using this technique, the sections of pavement were then stratified and assigned rating numbers of 1-12 as shown in Figure 3.

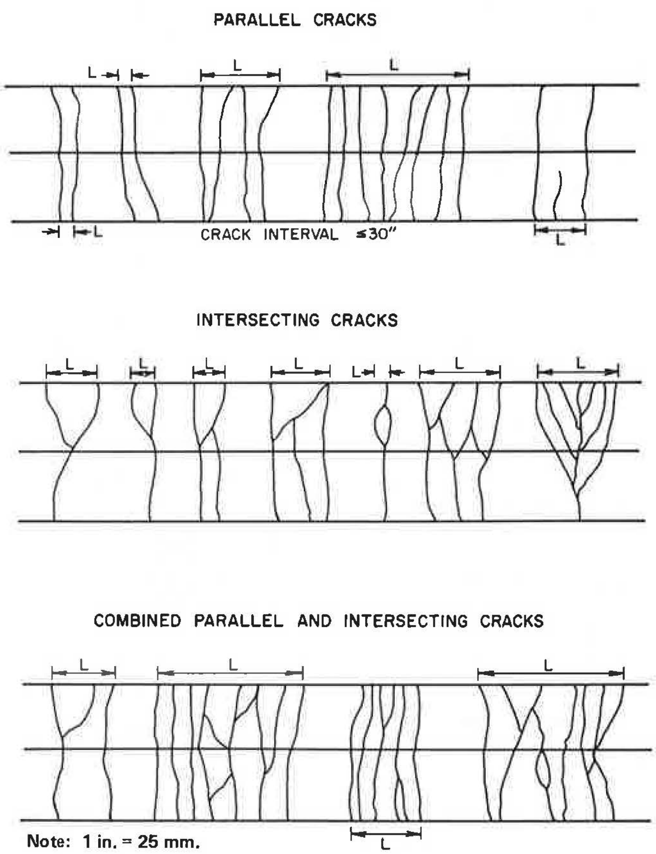
Selection of Maintenance Methods

The types of maintenance that were considered to be appropriate for the given rating numbers are given below. (This list was used as a "shopping list" from which maintenance types could be chosen to fit construction needs.) An attempt was made to apply as many types of appropriate maintenance as possible to the various ratings, but in some cases not all the types of maintenance were used.

Rating	Type of Maintenance
1	None
2	None, patch
3	None, patch

Rating	Type of Maintenance
4	None, underseal and overlay, underseal, overlay, concrete shoulders, drain
5	None; patch, underseal, and overlay; patch and underseal; patch and overlay; patch and concrete shoulders; patch and drain; patch, drain, and concrete shoulders
6	None; patch, underseal, and overlay; patch and underseal; patch and overlay; patch and concrete shoulders; patch and drain
7	None, drain, overlay
8	None, patch and overlay, patch and concrete shoulders, patch and drain, patch
9	None, patch and overlay, patch and concrete shoulders, patch and drain, patch
10	None, underseal and overlay, concrete shoulders, drain
11	None; patch, underseal, and overlay; patch and drain; patch and concrete shoulders; full-depth bituminous
12	None; patch, underseal, and overlay; patch and drain; patch and concrete shoulders; full-depth bituminous; patch, underseal, overlay, drain, and concrete shoulders; patch, drain, and concrete shoulders

Figure 2. Typical crack patterns.



Note: 1 in. = 25 mm.

Practical combinations of techniques that were possible are as follows:

1. Underseal prior to overlay,
2. Underseal prior to construction of concrete shoulders,
3. Install drains prior to overlay, and
4. Install drains prior to construction of concrete shoulders.

The list was compiled during a conference of personnel from Purdue University, the Indiana State Highway Commission, and the Federal Highway Administration.

Layout of Study Sections

The layout of study sections of a given maintenance type was governed by four criteria:

1. All sections were patched by using concrete as minimum maintenance. Hence, the performance of all sections started on a uniform basis, i.e., completely repaired pavement.
2. It was desirable to have a section of one type of maintenance be as long as possible.
3. At least one control section was retained for each rating number (1-12); these control sections were patched as needed.
4. As many different types of maintenance methods were used as possible for each rating number.

Figure 3. Rating of test sections.

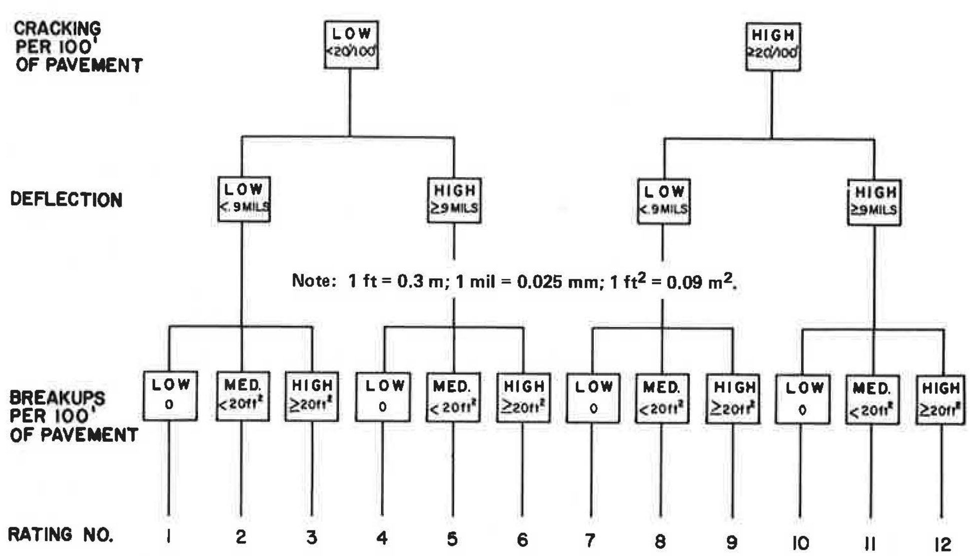
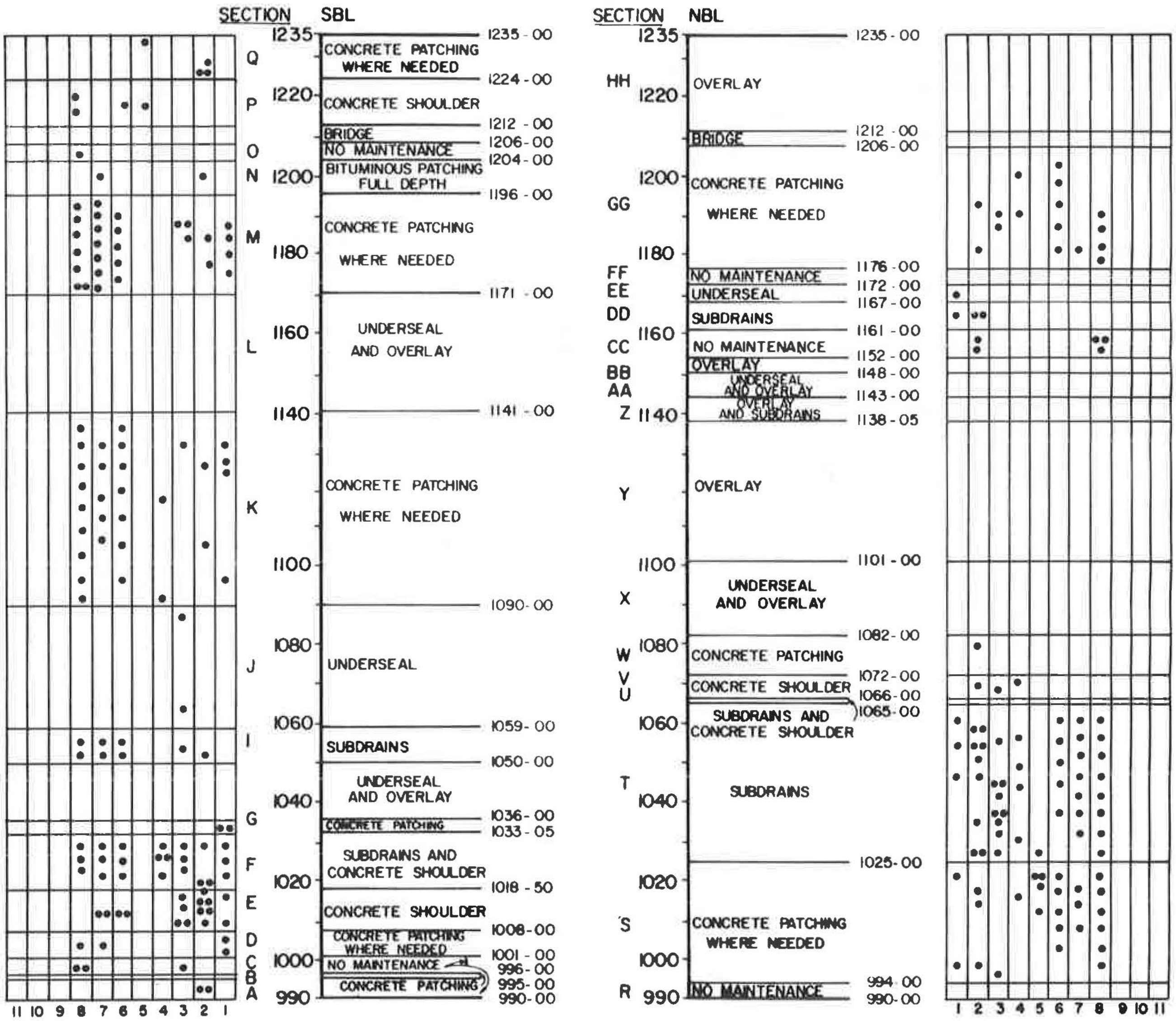


Figure 4. Failure as a function of location and time.



MAINTENANCE METHODS USED

The maintenance test sections were constructed by using standard techniques under the supervision of project engineers, as is normally done. Purdue personnel merely advised when needed on technical matters that arose during the construction.

All the breakups on the project were patched by using concrete regardless of the type of maintenance. The exceptions were those sections designated bituminous-patch sections, in which bituminous patches were used in lieu of concrete patches for the purpose of evaluating them, and the no-maintenance sections, which required no maintenance at all. Figure 4 shows the layout of the maintenance test sections as they were finally constructed.

Concrete Shoulders

The intent of using the concrete shoulders was to stiffen the pavement and thereby reduce deflection and subsequent pumping and failure of the pavement. The concrete shoulders were constructed so as to have contraction joints spaced at 4.5-m (15-ft) intervals. The slabs were 152 mm (6 in) thick on the outside edge and 229 mm (9 in) thick on the inside edge. They were tied to the existing pavement by using no. 4 tie bars 762 mm (30 in) long.

The tie bars were spaced at three different intervals on centers: 0.304 m (1 ft), 0.609 m (2 ft), and 0.762 m (2 ft 6 in). One section 365 m (1200 ft) long was tied into an existing keyway, whereas the remaining 0.609 m was tied to the vertical face of the pavement; thus, a butt joint was formed. The tie bars were omitted 0.762 m on either side of the construction joints to permit independent movement between the existing CRCP and the concrete shoulder.

After excavation and grading of the existing asphalt concrete shoulder, holes were drilled into the existing pavement to a depth of 355 mm (14 in) by using a tractor-mounted drill. The tie bars were grouted into the existing pavement by using a combination of epoxy and a nonshrinking grout.

Rumble strips were installed every 18.5 m (60 ft) along the concrete shoulder. Construction joints were installed at the end of each day's pour. These joints were always located at the middle of a slab and tied by using tie bars.

Subdrains

One of the major contributors to performance of this pavement was pumping, which resulted from free water on top of the subbase. For this project, subdrains were installed directly adjacent to the pavement edge. This was done in an effort to trap and remove water from the top of the subbase as quickly as possible and to drain infiltration water from between the pavement and the shoulder.

The backfill for the subdrains was carried up beyond the bottom edge of the pavement and to within 77 mm (3 in) of the top of the pavement. The top 77 mm was filled with asphalt concrete shoulder material.

Undersealing

Since pumping was considered one of the primary modes of failure for this pavement, undersealing (both with and without an overlay of asphalt concrete) was a major variable in the experimental layout. The underseal was intended to fill voids that might exist between the pavement and subbase.

The pavement was undersealed by using asphalt at specified locations. Holes of 0.07-mm (2-in) diameter were drilled through the pavement in the right-hand lane. After pumping had been completed, the hole was filled by a wooden plug.

When the undersealing operation first began, the proper spacing of the holes was not known. Hence, it was necessary to establish criteria that could be used by the contractor for spacing the holes and pumping the asphalt with a minimum of delay during the construction process.

In general, one row of holes spaced 2.4 m (8 ft) on centers was used at locations at which deflections were greater than 2.28×10^{-2} mm and there was uniform crack spacing. Two rows staggered every 1.2 m (4 ft) were used at locations that had high deflections associated with closely spaced cracking. At least two holes were drilled on each side of a potential failure or on each side of a potential patch.

If the deflection was less than 2.28×10^{-2} mm, the pavement was not undersealed. Hence, within each underseal section, some of the pavement was not undersealed, depending on the deflection value. Deflection was used as a criterion for undersealing to preclude creation of possible voids by lifting the pavement from the subbase.

Asphalt Concrete Overlay

The intent of the asphalt concrete overlays was to reduce deflection but, in addition, to perhaps prevent infiltration of water and thus improve performance. It was not known at the time of construction how thickness of overlay might influence performance, particularly from the standpoint of reflection cracking. Therefore, thickness of overlay was included as one of the variables of the experiment.

As can be seen from Figure 4, overlay was used on the southbound lanes in sections H and L. In the northbound lanes the primary overlay areas were in sections X and Y.

In general, a standard overlay thickness of 51 mm (2 in) was used, except as noted below. The existing pavement was overlaid by using a bituminous base 75 mm (3 in) thick on all locations except the longest overlay on the northbound lane (NBL) between stations 1082+00 to 1152+00 (sections X to BB). In this section an overlay wedge was formed by varying the base thickness as shown below (1 mm = 0.04 in):

NBL Station Limits	Thickness (mm)
1082+00-1110+00	51
1110+00-1119+00	76
1119+00-1129+00	127
1129+00-1152+00	76

Concrete Patching

As mentioned previously, concrete patching was required on all sections as the basic form of repair prior to the placing of any other type of maintenance. This is with the exception of section N, in which the basic experiment called for full-depth bituminous patching. The layout of the experiment required that any location in which breakup had occurred must be patched in addition to use of any other kind of maintenance.

In Figure 4, it may be noted that there were several no-maintenance sections in the experiment. These were sections of pavement that were considered to be in good repair. Hence, they were excluded from further analysis.

Full-Depth Bituminous Patching

Full-depth patching was included to compare the effect of bituminous patching with that of the concrete patching previously discussed. A minimal amount of full-depth patching was used. In addition, however, during the subsequent maintenance of the pavement, bituminous patches were placed if small failures existed.

Construction of the full-depth bituminous patches was similar to that of the concrete patches; however, all the existing steel within the patched area was removed along with the concrete to the pavement depth. After the subbase had been compacted, a no. 5 asphalt base was installed with a type-B surface, which brought the patch back to the existing grade.

PERFORMANCE OF TEST PAVEMENT

The maintenance test sections were completed in the fall of 1975. A complete performance survey was made immediately prior to construction of the test sections. The performance surveys were repeated immediately after construction of the test sections and then were repeated again each spring and fall until the fall of 1977. These surveys included complete logging of the extent of cracking that had occurred and of distress (punchouts) that had taken place and measurement of deflection by using the Dynaflect at 7.6-m (25-ft) intervals. Deflections were made by using only the sensor immediately under the vibrating wheels of the Dynaflect.

For the purposes of this paper, emphasis is placed on the effectiveness of each of the maintenance methods. Data relative to the cost-effectiveness of each of the methods will be presented.

The primary factor considered here is the occurrence of breakups (primarily punchouts) on each of the test sections and comparison of this occurrence with that on the corresponding control sections. It is to be recalled that each test section had a corresponding control section that had an identical original rating number determined by the criteria in Figure 3.

Progression of Breakup Occurrence with Time

Figure 4 shows the number of failures that have occurred on the test pavement during various time periods. This test pavement is considered to be a severe test of maintenance methods since it contained most of the features that have been known to contribute to poor performance of CRCP in Indiana.

Table 2. Summary of failure data.

Item	Failures per 100 m	
	Through Fall 1977	Through Spring 1979
Control section	0.79	1.656
Maintenance method		
Underseal and 51-mm overlay	0	0
Underseal and 76-mm overlay	0	0
51-mm overlay	0	0
76-mm overlay	0	0
127-mm overlay	0	0
Subdrain and 76-mm overlay	0	0
Underseal	0[a]	0[a]
Bituminous patch	0.41	0.94
Subdrain	1.73	3.281
Concrete shoulder	1.84	6.43
Subdrain and concrete shoulder	2.92	4.941

Note: 1 mm = 0.04 in.

[a]Several failures have occurred in the underseal section but not at locations actually undersealed.

(The numbered columns in Figure 4 relate to the periods of survey, as tabulated in the next paragraph.)

The occurrence of failures bore a general relationship to the season of the year. The data showed that a large portion of the failures occurred during fall, winter, and spring and that the occurrence of failures was reduced during the summer. As shown below, 65.8 percent of the failures occurred during fall, winter, and spring, and 34.2 percent occurred during spring and summer:

Survey	Period	Season	Total (%)
1	Aug. 1975-March 1976	Winter	10.5
2	March 1976-Aug. 1976	Spring-summer	17.5
3	Aug. 1976-Dec. 1976	Fall-winter	12.3
4	Dec. 1976-July 1977	Winter-spring	6.1
5	July 1977-Oct. 1977	Summer	3.1
6	Oct. 1977-April 1978	Fall-winter	15.7
7	April 1978-Oct. 1978	Spring-summer	13.6
8	Oct. 1978-June 1979	Fall-winter	21.2

Results of Failure Surveys

Solely on the basis of the occurrence of additional failures after construction of the test sections, the data indicated that the overlay sections performed better than any of the other test methods did. A summary of failures that have occurred since construction of the test sections is listed in Table 2. In this table, the data are arranged in descending order relative to number of failures per 100 m.

It is apparent from these data that the concrete shoulders were not effective in this particular experiment nor were the subdrains. A large portion of the concrete-shoulder sections were concentrated at the southern end of the southbound lanes at which poor performance had been demonstrated prior to the experiment; hence the data may have been influenced by factors other than the use of shoulders themselves.

The subdrains that were installed in this particular test project were relatively deep and are no doubt draining the subgrade as well as the subbase. The back-filling operation adjacent to the pavement edge was always carried up to the mid-depth of the concrete pavement, and therefore any water that lies on the subbase should go through the drains. It is known that the drains are functioning. However, the subbase under the pavement is impervious and contains a high percentage of fines. Therefore, rapid percolation of the water through the subbase itself is probably not taking place, and any water that is drained from the subbase is that which lies at the interface between pavement and subbase.

COST ANALYSIS

The cost analysis in this study consisted of an evaluation of the cost-effectiveness of the various methods. These data are summarized in Table 3. It may be seen that the overlay method was very effective in stopping the progression of failures but, except for the concrete-shoulder sections, it was the most costly.

The underseal appeared to be a cost-effective means of at least slowing down the progression of failures, since no failures have been found in those sections that have actually been undersealed.

The subdrain and concrete-shoulder sections were found to be the least cost-effective means of stopping progression of failures. It is to be recalled, however, that the concrete shoulders and subdrains were used mainly at the extreme southern end of a construction job on which an initial high incidence of failures had taken place prior to construction of the test pavements.

Construction Costs Compared with Patching Costs

Table 3 illustrates that only seven of the methods (the first seven listed) were effective in reducing or completely stopping occurrence of failures in the test pavement. It may be seen that all but the last of these includes an overlay of asphalt concrete. The last method is undersealing without the use of an overlay. In the analysis that is presented in Table 4, the last four methods in Table 3 and the control section are excluded.

For the data in Table 4 it is assumed that all the failures were patched during the life of the test, although all the failures were not necessarily patched. It is further assumed that failures that occurred during the summer, fall, and winter months would be repaired during the next construction season. The cost of patching is given in 1975 dollars, and the average cost of patching for this experiment ($1515 per patch) is discounted at the rate of 6 percent.

The data indicate that the overlay, although very effective in stopping failures, was not cost-effective during the period of the experiment when cost of constructing the maintenance was compared with the average cost of maintaining the pavements (last row of table). The underseal sections were constructed at an average cost of $1020/100 m ($311/100 ft). This technique slowed down occurrence of failures. The data suggest that undersealing is a viable method for slowing down the occurrence of distress in pavements of this type.

PERFORMANCE SUMMARY

Three types of maintenance were investigated in this research project other than the usual concrete patching done on a routine basis in the state. Various combinations of these maintenance techniques were used, and variations of the methods were included in some cases. The basic methods include the following: (a) overlay by using asphalt concrete both with and without drainage and undersealing, (b) installation of subdrains at the outside edge of the pavement, and (c) concrete shoulders.

It is pertinent to note first that the test pavement was already among the worst-performing pavements in the state at the time the experiment was set up. Because of the high incidence of failures in the test pavement prior to establishing the main-

Table 3. Summary of performance data.

Item	Length (m)	Cost[a] ($/100 m)	Failures per 100 m Through Fall 1977: Maintenance Section	Failures per 100 m Through Fall 1977: Control Section	Failures per 100 m Through Spring 1979: Maintenance Section	Failures per 100 m Through Spring 1979: Control Section
Control section	6214	2 385	-	0.787	-	1.656
Maintenance method						
Underseal and 51-mm overlay	853	4 771	0	1.788	0	3.281
Underseal and 76-mm overlay	1219	6 772	0	0.731	0	1.916
51-mm overlay	274	4 351	0	0.862	0	1.043
76-mm overlay	1373	6 404	0	1.082	0	1.906
127-mm overlay	304	6 598	0	1.289	0	2.46
Subdrain and 76-mm overlay	150	7 149	0	0.935	0	1.873
Underseal	1097	1 020	0[b]	0.397	0[b]	1.036
Bituminous patch	243	3 780	0.41	0.82	0.935	1.886
Subdrain	1676	3 307	1.729	1.007	3.281	2.083
Concrete shoulder	870	9 876	1.837	1.115	6.43	1.902
Subdrain and concrete shoulder	443	11 519	2.92	0.935	4.941	1.4

Note: 1 mm = 0.04 in.

[a]Includes cost of patching before construction of maintenance section.
[b]Three failures have occurred in the underseal section at locations that were not undersealed.

Table 4. Cost of maintenance sections and potential cost of patching control sections.

Maintenance Method	Cost of Maintaining Pavements ($/100 m): Maintenance Section	Control Section: Year 1	Year 2	Year 3	Year 4
Underseal and 51-mm overlay	4711	649	1709	2792	4209
Underseal and 76-mm overlay	6772	377	990	1653	2483
51-mm overlay	4351	203	538	882	1335
76-mm overlay	6404	374	990	1653	2483
127-mm overlay	6598	485	1260	2093	3156
Subdrain and 76-mm overlay	7149	364	971	1591	2401
Underseal	1020	203	538	879	1325
Avg	-	380	997	1647	2483

Notes: Patching costs are based on the assumption that all failures are patched and at an average cost of $1515/patch. Costs are accumulated with time.
Patching costs are in 1975 dollars discounted at 6 percent.
Time is measured in years after completion of construction in fall of 1975.
Added user costs during patching are not considered.

tenance experiment, it can be argued that the results of the experiment itself have been influenced by the initial condition of the pavement. In other words, some of the techniques that were investigated in this experiment that did not prove successful might have shown better performance if the pavement had not already deteriorated to the extent that it had prior to establishment of the experiment. Doubtless, if the pavement had been only slightly in distress, some of the techniques investigated could have acted as preventive maintenance, and the results may have been somewhat different.

In the performance summary that follows, the methods are given in the order of most effective to least effective in preventing further deterioration of this specific pavement regardless of cost. The reader is referred to Table 4 in which the cost-effectiveness of each of the methods is presented.

1. Asphalt overlay: The asphalt overlay sections have performed very well and no correlation has been found between the performance of the overlay and its thickness. No reflection cracking has been noted other than on the feather sections, in which there is a transition from the overlay to a pavement that has no overlay, and at two isolated locations at which transverse cracks have been seen over known concrete patches.

2. Underseal and overlay: These sections have performed very well and no additional failures have occurred.

3. Overlay and subdrain: Again, the pavements that had an overlay have shown excellent performance, and the data do not suggest that the installation of subdrains along with the overlay has aided substantially in the performance of the pavement.

4. Underseal: Performance of the underseal sections has been quite satisfactory. It is to be recalled that undersealing was actually done only at locations at which the deflection was greater than 2.3×10^{-2} mm as measured by the Dynaflect. No failures have occurred where undersealing was done, although three have occurred in the underseal sections at locations other than those that were undersealed. The undersealing was done at a cost of $1020/100 m, and the data suggest that this is a cost-effective method of preventive maintenance.

5. Bituminous patch: The bituminous patches have shown no distress, and no failures have been found associated with the patches. Some settlement has occurred and some roughness is noted in riding over the patches.

6. Subdrain: Numerous failures have occurred on the sections in which subdrains were installed. The subdrains are known to be removing water from under the pavement as shown by a decrease in pumping and by visual inspection of the drain outlet. However, the numerous failures in these sections have suggested that additional work could well be done to determine the best means of draining pavements of this type.

7. Concrete shoulder: The concrete shoulders have not performed well, and deflection measurements suggest that continued distress might occur. One of the test sections included a keyway at the pavement edge, whereas the others did not. The keyed section has performed very well, but the unkeyed sections have not.

In the decision-making process it is necessary to evaluate performance of pavements in light of the economics of the problem. For this case, the primary question asked was what the most effective means of maintaining CRCP was.

For this experiment, overlaying by using asphalt concrete completely stopped further deterioration of the pavement. However, the average cost of patching during the time span of this test was $2483/100 m ($757/100 ft) for the control sections, whereas the initial construction costs of overlay were higher in all cases--$4351/100 m ($1326/100 ft) for the 51-mm

(2-in) overlay. Hence it can be concluded that, solely from the standpoint of cost, overlay is not cost-effective. Further, neglecting inconvenience and safety during patching operations, a cost-effective means would be to merely let breakups occur and to patch them as needed.

Undersealing also halted further deterioration and was done at a cost of $1020/100 m, which was less than the cost of maintaining the control sections. Hence it was concluded that this was the most cost-effective means of maintaining this pavement. The primary mode of distress for this pavement was pumping caused by high deflections and an impervious subbase. Undersealing effectively corrected this defect.

For this analysis it was assumed that all breakups would be repaired during the next summer if the failure had occurred during the winter or spring or during the same summer if the failure had occurred during the summer. It is known that all the breakups were not patched by using concrete. In some cases, temporary measures were taken, and the breakup still exists on the test pavement. Hence the economic analysis reported here is conservative.

As in any engineering project, the decision relative to type of maintenance to be applied must consider all factors and not merely cost. For this particular pavement the question of the desirability of expending large sums of money for complete removal of the possibility of further distress is moot.

Perhaps the primary conclusion to be reached from studies of this type is that, to be effective from the point of view of both performance and cost, maintenance procedures must be evaluated in the light of the defects that are to be corrected and planned accordingly. It is axiomatic that in an energy-constrained environment, design and maintenance should be carefully evaluated and based on sound engineering principles.

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Patching Jointed Concrete Pavements

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The experiences of the Virginia Department of Highways and Transportation in the repair of jointed concrete pavements over the past 10 years are summarized. Persons involved in pavement repair are cautioned to give careful consideration to pavement geometrics and dynamics. Also emphasized is the need for proper consolidation, adequate quality-control procedures, and care in the selection of repair materials. The conclusion is that serviceable repairs to concrete pavements can be achieved if these factors are given full consideration.

In the repair of jointed concrete pavements, the choice of the procedure to be used must include a consideration of how the pavement responds to its environment. The two major components of this environment are traffic loadings and climatic conditions.

The response of the pavement to traffic loadings is within the purview of the design activity and will not be considered at length here. It is evident, however, that when a pavement no longer provides the service for which it was designed, the selected maintenance procedure should restore it to the original level of service. For example, when pavement support conditions change so that wheel loads become more destructive, the maintenance engineer must attempt to restore the original support conditions through undersealing, slab-jacking, and improving the drainage.

The response of the pavement to climatic conditions also is treated in the design analysis, in which curling, warping, and joint movements are considered. Maintenance personnel, however, generally seem to be much less aware than designers are that the pavement is more or less in constant motion and that service conditions and repair techniques must make allowances for this motion. As a result of this lack of awareness, one often sees repairs that are incompatible with service conditions, e.g., unjointed patches on jointed pavements.

Instances in which pavement movement of one type or the other must be accommodated will be pointed out in succeeding sections of this paper. For now, it is sufficient to emphasize that pavements are dynamic and that, whether one is dealing with design, construction, or maintenance, those dynamics must be considered. From the standpoint of pavement repairs, one can state simply that the repair must be geometrically compatible with the dynamic pavement system.

DISTRESS MECHANISMS AND MANIFESTATIONS

Joint failures of various types probably lead to the necessity for much more maintenance than do problems with the slabs. Numerous factors can contribute to joint failures, and the failures can be manifested in several ways. Because the repair procedure to be used is often dependent on the nature of the distress, some of the mechanisms and manifestations of joint failures are discussed below.