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16. Abstract

The behavior of a plain, 14-in. portland cement concrete (PCC) pavement with short, randomly-spaced (13-, 19-, 18- and 12-ft), skewed contraction joints and without a subbase is being compared to an 8-in. continuously reinforced concrete (CRC) pavement overlying a 4-in. bituminous aggregate mixture (BAM) subbase. Internal curing temperature, joint width, faulting, cracking, and smoothness observations have been summarized. Initially, the CRC pavement was smoother than the plain PCC pavement, mainly because construction equipment and procedures differed. So far, the serviceability of both pavements remains high, and no trends have developed because neither pavement has developed any observable distress. At contraction joints, corner deflections were higher where load transfer depends on aggregate interlock rather than dowel bars. Since mean joint openings have exceeded 0.035 in. to 0.040 in., aggregate interlock, according to other research, cannot serve as an effective means of load transfer. Yet, development of joint faulting has been delayed because of other factors such as low traffic volume, joint skewness, pavement thickness, and subgrade support.

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DEPARTMENT OF TRANSPORTATION
Bureau of Materials and Physical Research

BEHAVIOR OF PLAIN PCC PAVEMENT WITH SKEWED JOINTS RANDOMLY SPACED

By

Lloyd J. McKenzie

Final Report
IHR-506
Behavior of Plain PCC Pavement

A Research Study Conducted by
Illinois Department of Transportation

in cooperation with
U. S. Department of Transportation
Federal Highway Administration

June 1978

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BEHAVIOR OF PLAIN PCC PAVEMENT WITH SKEWED JOINTS RANDOMLY SPACED

INTRODUCTION

This research was undertaken to obtain new knowledge about the behavior of extra-thick, non-reinforced, portland cement concrete (PCC) pavement placed on the earth subgrade relative to that for continuously reinforced concrete (CRC) pavement placed on a stabilized subbase under similar traffic and environment. This experimental pavement is unique to the Division of Highways in several respects. First, the contractor who was awarded the work opted to combine a 10-in.-thick pavement and a 4-in.-thick PCC subbase into a 14-in., extra-thick, non-reinforced PCC pavement placed directly on the subgrade by a slipform paver. Second, sawed transverse contraction joints without load-transfer devices were cut in the pavement at a random repeating pattern of 13 ft, 19 ft, 18 ft, and 12 ft (3.96 m, 5.79 m, 5.49 m and 3.66 m) to eliminate the tendency in some vehicles to vibrate in resonance at certain speeds during passage over the pavement. Third, the transverse joints, instead of being cut normal to the pavement edge, were skewed counterclockwise a distance of 4 ft (1.22 m) in 24 ft (7.32 m) of pavement width, thereby allowing a vehicle axle to pass over the joints one wheel at a time.

For this study, pavement behavior is defined as any change in riding quality, serviceability level, and pavement surface condition with time and traffic. As soon as the pavements were completed and were opened to traffic, the collection of the necessary data to evaluate their behavior was begun.

Construction of the experimental pavement was completed in 1973, but the continuously reinforced pavement that serves as the control section was not completed until 1975. Thus, data are available for 3 years from the experimental pavement and for only 1 year from the CRC control pavement. This short service life is insufficient for any significant trends in pavement behavior to have developed and, since traffic is light, observable differences are unlikely to appear for several years.

Since December 1977 is the completion date for this study, this report not only describes the experimental details, construction details and field measurements but also summarizes field data and discusses appropriate findings to date as well as offers recommendations.

EXPERIMENTAL DETAILS

The experimental plain PCC pavement extends west from the west terminus of the East-West Tollway (Route Illinois 5) at its intersection with Route U. S. 30 for 8.087 mi. (13.01 Km), paralleling the south city limits of the City of Rock Falls, Whiteside County, Illinois (Figure 1). This four-lane divided highway has a depressed median and full-depth bituminous aggregate mixture (BAM) shoulders, which are 10 ft (3.05 m) wide on the right-hand side and 6 ft (1.83 m) wide on the left-hand side. Subsurface drainage is provided by 6-in. pipe underdrains (Figure 2) along each edge of the pavement which is 24 ft (7.32 m) wide, and 14 in. (356 mm) thick placed directly on a prepared subgrade. Neither reinforcement nor mechanical load transfer at the joints was provided in the pavement except for one special test section where dowel bars were installed. The transverse contraction joints, as previously mentioned, were sawed at a varying interval of 13 ft, 19 ft, 18 ft and 12 ft (3.96 m, 5.79 m, 5.49 m and 3.66 m) which was repeated, and they were skewed counterclockwise 4 ft (1.22 m) in 24 ft (7.32 m) (Figure 3). After sawing, the joints were sealed with a hot-poured asphalt joint sealant.

The control pavement, which extends 10 mi (16.1 Km) west of the experimental pavement, lies between Route Ill. 2 and Ill. 78 near Denrock, Illinois. The control section also is a four-lane divided highway with a depressed median and with full-depth BAM shoulders; however, the slab is an 8-in. CRC pavement placed on a 4-in. BAM subbase. The subbase was placed wide enough to furnish paver track support outside the pavement edge. Subsurface drainage was provided by 6-in. pipe underdrains under the shoulders along each edge of the BAM subbase.

The experimental pavement, which was opened to traffic in November 1973, is almost 2 years older than the control section, which was opened to traffic in September 1975.

Test Sections for Joint-Opening Study

The experimental pavement contains four test sections each containing 12 consecutive transverse joints. One test section has load-transfer dowels in each joint and can be seen in the foreground of Figure 1. The remaining three test sections have no mechanical load transfer in the joints and are representative of the entire experimental pavement. The following table lists the location of each test section together with its length and with the method of load transfer.

TABLE 1. LOCATION OF TEST SECTIONS

Section No.	Location	Traffic Direction	Length (ft)	Load Transfer
A	2583+40 to 2585+02	WB	162	Aggregate Interlock
B	2615+04 to 2616+70	EB	164	Aggregate Interlock
C	2355+13 to 2356+80	EB	167	Dowels
D	2368+26 to 2369+98	WB	172	Aggregate Interlock

Astride each joint in the test sections, brass gage plugs were cemented securely into holes that had been drilled into the concrete. The gage plugs, 10 in. (254 mm) apart, straddled the joint in the center of the traffic lane on a line paralleling the pavement centerline. When the gage plugs were installed, their initial spacing was measured to the nearest one ten-thousandth of an inch, and the difference between the initial and subsequent measurements may be interepreted as opening and closing of the joints that is caused by thermal changes in the pavement. The measurements were made with a Whittemore strain gage, which was firmly seated in 0.125-in. (3-mm) holes that had been drilled in the brass plugs. Between measurements, rivets filled the holes to keep them clean.

Internal Slab Temperature

Internal pavement slab temperatures were measured from 1-hr to 4-hr intervals for 30 hrs following placement of the plastic concrete. Thermocouples were installed at 0.5 in. (13 mm), 7 in. (178 mm) and 13.5 in. (343 mm) below the paved surface at two places. To maintain the correct depth with respect to the surface, the thermocouples were inserted in a 0.5-in. diameter plastic rod, which was pushed into the subgrade. The thermocouple leads were placed in a shallow trench, which had been cut in the subgrade, and they were covered with soil for protection during paving. At the outside edge of the slab, the leads were coiled and were buried in the subgrade. After paving, the leads were mounted on marker stakes, which were easily accessible for measuring the slab temperatures.

CONSTRUCTION DETAILS

Both the control and the experimental sections were placed by the slipform method. The control section was placed using a Blaw-Knox paver traveling on the BAM subbase. Slab thickness was controlled by 10-ft ski-sensors also riding on the subbase. The 14-in. experimental pavement, on the other hand, was placed on the earth subgrade with a CMI Autograde slipform paver (Figure 4), which was equipped with automatic slab-thickness control from multi-foot skis traveling on the subgrade between the paver track and the trailing slipform (Figure 5). With this paver, mortar carried by the screeds continually dropped onto the subgrade in front of the skis and interfered with grade control. To eliminate this interference, the mortar had to be cleared away from the skis.

Initially, the transverse contraction joints were sawed consecutively with a single-blade portable saw. Soon after paving started, this method was discontinued in favor of first sawing every fourth joint, and then sawing the remaining joints. This procedure lessened the possibility of uncontrolled cracking. Later, the joints were sawed consecutively with a four-blade saw which could keep pace with the paving.

FIELD MEASUREMENT DETAILS

Pavement smoothness was measured with the Illinois Roadometer (4) before the road was opened to traffic to establish its initial Present Serviceability Index (PSI). Subsequently, in 1975 and in 1976, measurements were repeated during May.

In the summers of 1974 and 1976, a condition survey of the experimental pavement documented the extent of pavement cracking as well as other forms of pavement distress. Crack development, after paving, was monitored closely in four 1000-ft sections, each containing one of the four test sections. Faulting at the joints in each test section also was measured with the fault gage shown in Figure 6.

Summer (August) and winter (January) joint openings were measured with a Whittemore strain gage. Measurements were made in 1973, 1974, 1975 and 1976. At the same time, air temperatures in the shade were recorded.

In May 1974, static rebound deflections under an 18-kip single-axle load were measured at one corner and at the outside edge of each pavement panel in the test sections.

Traffic surveys, conducted regularly by the Division of Highways, were used to compute cumulative 18-kip equivalent single-axle loads.

FINDINGS

In this section the data summaries are presented, the significance of the data is discussed, and interpretations are made.

Internal Concrete Curing Temperature

The internal temperature of the experimental 14-in. pavement slab was measured from 1-hr to 4-hr intervals near the surface (0.5-in. depth), in the center (7-in. depth), and near the subgrade (13.5-in. depth), beginning when the concrete was placed and ending 50 hrs later. The first measurements were obtained in a ramp wedge in May, and the second set was obtained in the eastbound pavement slab in June. Plots of the internal slab temperatures for each slab depth at both locations and for equivalent air temperatures are charted in Figure 7. At the time of placement, the soil temperature was $62^{o}F$ ($17^{o}C$) in May and $77^{o}F$ ($25^{o}C$) in June.

As can be seen in the figure, on both dates maximum temperatures were reached first near the top of the slab, 7 to 9 hrs after the concrete was placed. The peak in the center of the slab occurred between 11 and 12 hrs after placement, followed by the lower part of the slab 16 hrs after placement. The slab temperatures still were well above the ambient temperature 30 hrs after placement. It is also of interest that the slab temperature at the surface was most influenced by changes in ambient temperature, followed by the middle depth, while the ambient temperature had little if any effect at the bottom of the slab.

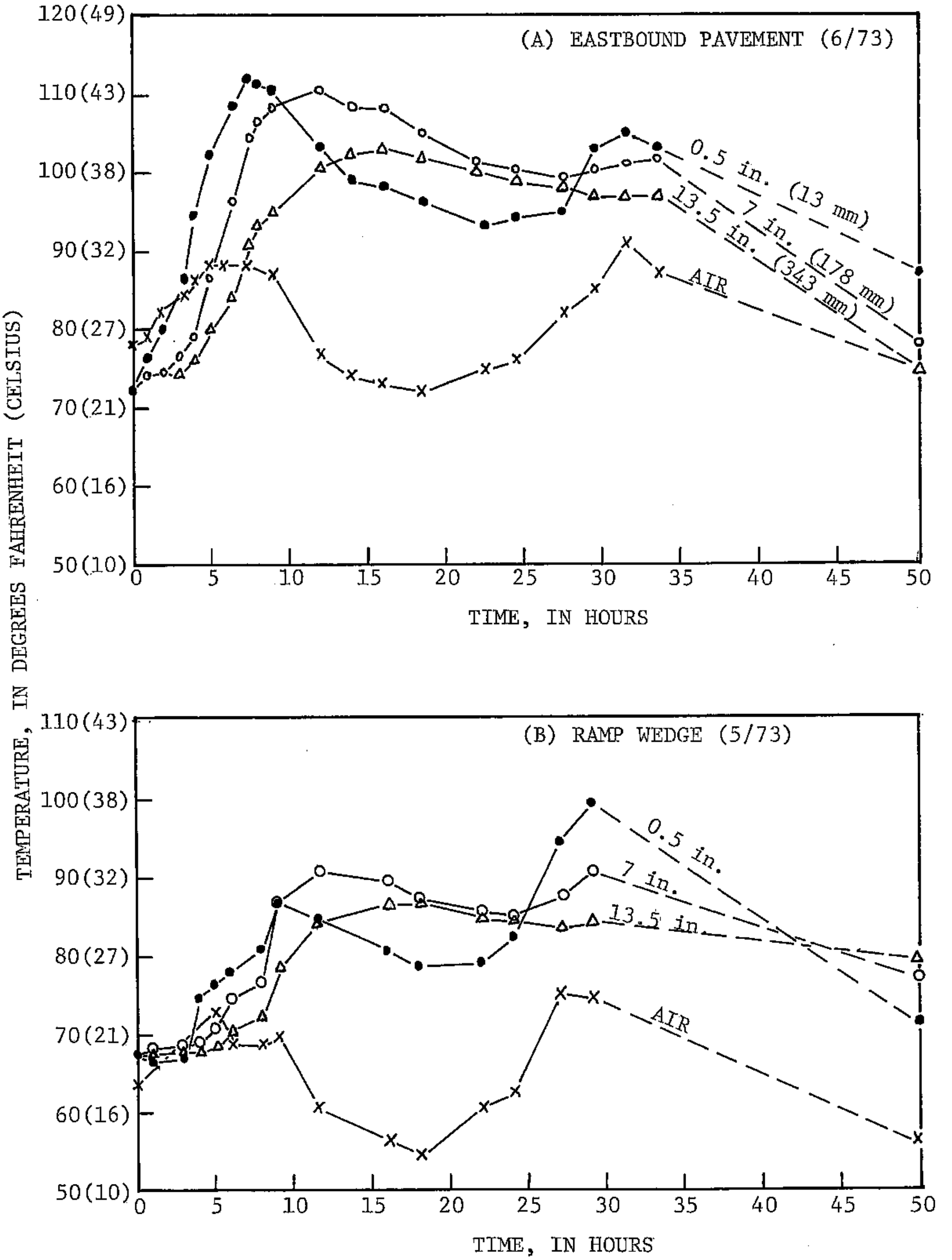

Figure 7. Internal curing temperature in 14-inch PCC Pavement.

However, both the time to reach the maximum temperature and the magnitude of that temperature were influenced strongly by ambient temperature, soil temperature, air movement, and cloud cover. During the May placement the sky was overcast, and the air temperature was in the low 60's. During the June placement, however, the sky was clear and sunny, and the air temperature was in the high 70's.

Static Rebound Deflections

Static rebound deflections were measured (May 1974) with the Benkelman beam at one outside corner and at the mid-panel edge of each pavement panel in the test sections. The average corner and the average edge deflection in each test section are tabulated below:

TABLE 2. AVERAGE STATIC REBOUND DEFLECTIONS IN TEST SECTIONS

Test Section	Type of Load Transfer	Avg. Deflection (inches) Corner	Edge
A	Aggregate Interlock	.014 (.36 mm)	.011 (.28 mm)
B	Aggregate Interlock	.016 (.41 mm)	.010 (.25 mm)
C	Dowels	.012 (.30 mm)	.010 (.25 mm)
D	Aggregate Interlock	.015 (.38 mm)	.010 (.25 mm)

As can be seen in the table, the edge deflections were uniform and relatively low (.010 in., 25 mm). Corner deflections as expected were somewhat higher than the edge deflections, but they varied with the type of load-transfer system that had been employed. The corner deflections were lowest where dowel bars were employed as the load-transfer system.

Pavement Condition Surveys

The first condition survey of the experimental pavement was conducted in May 1974. At that time, the pavement surface of the four 1000-ft sections was examined carefully for transverse cracking, spalling, corner breakage, and other pavement distress.

A second condition survey was completed in August 1976. At that time no cracks of any kind were found in any of the surveyed sections, and the amounts of other distress such as edge spalls, corner breaks, and joint spalls were insignificant.

Faults measured in 1974 and in 1976 indicated no difference in faulting could be associated with use or nonuse of the doweled load-transfer system.

The manner in which the cracks formed at the sawed joints was observed closely, particularly in the test sections. In Section A, the joints first were examined when the pavement was 11 days old, and in Section B they were examined 3 days after placement, when the gage plugs were installed. In both sections, cracks had formed at some but not at other joints. In Section A, cracks had formed in the joints before the gage plugs were installed. In both Sections A and B, every fourth joint had been sawed before the remaining joints were sawed. Consequently, cracks first developed at every fourth joint rather than at intermediate joints. After the joints were sealed, summer temperatures caused the pavements to expand, and joint sealant was extruded from joints that had opened before the sealant was applied but not from uncracked joints.

In test Sections C and D, the joints were sawed consecutively. Gage plugs were installed at joints in C before any cracks had formed and in D after some cracks were partly formed. In these sections, crack formation at the joints occurred in a more uniform manner, and large joint openings did not develop before the gage plugs were installed and before the joints were sealed.

Nevertheless, transverse cracks did develop in all joints before the shoulder pavement was placed. Much of the cracking occurred after the contractor's vehicles were permitted on the new pavement. The absence of transverse cracks at the joints, where the gage plugs for measuring joint widths were installed, affected changes in joint openings.

Joint-Width Measurements

Beginning in August 1973 and ending in August 1976, the width of 12 consecutive joints in each test section was measured in summer when ambient temperatures were high, causing pavement expansion, and in winter when the temperature was low, causing pavement contraction. The average change in joint width from summer to winter (opening) and from winter to summer (closing) is listed in Table 3. The values in the table are inches of opening or closing from one measurement date to the next. The average joint-width change is for 12 consecutive joints. The range in joint-width change among the 12 joints also is tabulated in the table.

Each test section includes 11 pavement panels, three panels at 12 ft (3.66 m), 13 ft (3.96 m), and 19 ft (5.79 m) and two panels at 18 ft (5.49 m). The joint-width changes between 12- and 13-ft panels and between 18- and 19-ft panels also are given in Table 3.

As can be seen in Table 3, the average joint-width change (opening and closing) was .044 in. (1 mm) in the 1973-74 period, .051 in. (1.3 mm) in the 1974-75 period, and .031 in. (0.8 mm) in the 1975-76 period. These changes can be compared to mean temperature differentials of 60^{o}F (33.3^{o}C) in 1973-74, 61.5^{o}F (34.1^{o}C) in 1974-75 and 30^{o}F (16.7^{o}C) in 1975-76. Although the joint width between the 18- and 19-ft panels was wider than joint width between the 12- and 13-ft panels during the first year, the difference between shorter and longer panels by the third year was nearly the same. After 3 years of observations, Sections A, C and D respectively had net increases of .012 in., .006 in. and .024 in. (0.30 mm, 0.15 mm, 0.61 mm), while Section B had a net decrease of .008 in. (0.20 mm).

TABLE 3. AVERAGE CHANGE IN JOINT WIDTH IN THE SPECIAL TEST SECTIONS

Section No.	Movement	Average Joint-Width Change (inches) 1973-74 Average	1973-74 Range	1974-75 Average	1974-75 Range	1975-76 Average	1975-76 Range
A	Opening	.056	-.005 -- .152	.050	.023 -- .070	.037	.023 -- .052
	Closure	.049	-.076 -- .152	.057	-.002 -- .093	.024	.005 -- .033
B	Opening	.044	.019 -- .072	.048	.023 -- .066	.033	.017 -- .124
	Closure	.048	.016 -- .071	.055	.032 -- .076	.030	.015 -- .122
C	Opening	.042	-.003 -- .087	.049	-.004 -- .091	.029	.011 -- .048
	Closure	.038	-.005 -- .076	.058	-.003 -- .093	.026	.007 -- .044
D	Opening	.038	-.036 -- .091	.040	-.003 -- .082	.038	.005 -- .063
	Closure	.040	-.003 -- .083	.051	-.033 -- .091	.028	0 -- .051
Overall Ave.	Opening	.045		.047		.034	
Overall Ave.	Closure	.044		.051		.027	
12, 13 ft panels							
	Opening	.013		.038		.033	
	Closure	.017		.037		.024	
18, 19 ft panels							
	Opening	.043		.048		.035	
	Closure	.041		.059		.028	

Note 1: Each test section contains pavement panels that are 12, 13, 18 and 19 ft long.

Note 2: The joint openings were associated with a 60-degree temperature differential in 1973, '74 and '75 and a 30-degree differential in 1976.

Note 3: 1 in. = 25.4 mm

Assuming a coefficient of thermal expansion for paving concrete of 5.7 x 10^{-6} inch per inch of pavement length per degree Fahrenheit and a 60-degree temperature differential between summer and winter mean temperatures, the joint opening between 13-ft pavement panels would be .053 inches (1.35 mm) and that between 19-ft panels would be .078 inches (1.98 mm). The observed openings were less than the computed values, and this difference can be accounted for by subgrade drag.

Test Section C is equipped with a dowelled load-transfer system, but Sections A, B and D are dependent on aggregate interlock for load transfer across the joints. Colley and Humphrey, in 1967 (2), showed that the effectiveness of aggregate interlock in plain, jointed pavements depends on foundation bearing strength, aggregate size and shape, axle weights, slab thickness, aggregate attrition as related to load applications, and joint openings. Aggregate interlock is more effective when a stabilized subbase is used in place of a granular subbase, and angular aggregates, such as crushed gravel and crushed stone, are more effective in load transfer than rounded gravel aggregates. Other research (3) indicated that aggregate interlock, as a load-transfer system, becomes ineffective when joint openings exceed .035 to .040 inches (.90 to 1.0 mm). The above findings suggest that in the Rock Falls area, where the difference between summer and winter temperature can exceed $50^{\circ}F$ ($10^{\circ}C$), aggregate interlock will not be an effective load-transfer system.

At this time, the absence of any faulting at the joints in test Sections A, B and D as compared to Section C is most likely due to factors like pavement thickness, subgrade, joint skewness, and low traffic volume rather than to aggregate interlock as a load-transfer system.

Pavement Smoothness

The smoothness of the plain, jointed PCC pavement and the CRC pavement was measured with the Illinois Roadometer before they were opened to traffic (4). Smoothness measurements were repeated in 1974 and 1976, and Roughness Index values (in./mi) are presented in Table 4:

TABLE 4. ROUGHNESS INDEX OF PLAIN PCC PAVEMENT AND CRC PAVEMENT

Construction Section	Pavement Type	Direction	Ave. Roughness Index (in./mi) 1973	1974	1975	1976
195-3	Plain PCC	EB	88	82	-	77
		WB	90	84	-	78
195-2	CRC	EB	-	-	51	47
		WB	-	-	50	49

As can be seen in the table, the CRC pavement initially was smoother than the PCC pavement. Both pavements were smoother in 1976 than when they were constructed. This behavior, by experience, has been found to be normal for Illinois pavements early in their service life as they adjust to the environment and to traffic. As pavements age, pavement cracking, faulting, slab misalignment and other deterioration cause an increase in pavement roughness, which reduces the level of serviceability.

The present serviceability level of portland cement concrete pavement is given by the following equation (5):

$$PSI = 12.0 - 4.27 \text{ Log } RI - 0.09 \sqrt{C+P}$$

where:

PSI = Present Serviceability Index

RI = Roughness Index (Illinois Roadometer)

C = lineal feet of cracking in 1000 sq ft of pavement

P = square feet of patching in 1000 sq ft of pavement

By the equation, the PSI of a pavement is determined by the Roughness Index (RI) and by the cracking and patching in the pavement, but since the plain, jointed pavement has neither cracking nor patching, the PSI is determined by only the RI value. The pavement cracks in continuously reinforced concrete pavement are held together by the reinforcing steel. They develop early, are present throughout the service life of the pavement, and are considered to be an integral part of the pavement. As such, these cracks do not contribute to a loss in pavement serviceability unless they become enlarged, which indicates a failure in the reinforcing steel. Because no such failures exist in the CRC section, the PSI of the CRC pavement also was determined by only the Roughness Index. The PSI of both the experimental and the control section is listed in Table 5:

TABLE 5. PRESENT SERVICEABILITY INDEX OF THE EXPERIMENTAL AND OF THE CONTROL SECTIONS

Construction Section	Pavement Type	Direction	PSI 1973	PSI 1974	PSI 1975	PSI 1976
195-3	Plain PCC	EB	3.70	3.83	-	3.95
		WB	3.66	3.78	-	3.92
195-2	CRC	EB	-	-	4.71	4.86
		WB	-	-	4.75	4.78

As can be seen in the table, the CRC control pavement had a higher initial PSI than the PCC experimental pavement. This higher PSI corresponds to the lower initial RI and is attributed to construction procedures rather than to pavement behavior. Aside from the variation in initial smoothness, both pavement sections have become slightly smoother since they were placed. In turn, this has resulted in a slightly higher serviceability level. To date, no loss in serviceability that can be attributed to structural deficiencies has occurred in either of the pavement sections.

Traffic

From the 1975 Traffic Map for Illinois (1), the average daily traffic (ADT) on the experimental pavements was 1650 vehicles per day. Of the total traffic, 1000 vpd (61 percent) are passenger cars, 300 vpd (18 percent) are single-unit trucks, and 350 vpd (21 percent) are multiple-unit trucks.

Section 7 in the Highway Design Manual (6) gives the procedure for computing a traffic factor, which is the total number of 18-kip equivalent single-axle loads (ESAL), in millions, that will be applied during the design period. Using this procedure and the total ADT for 1975 as the design traffic, the total number of 18-Kip ESAL applications that will be applied during the design period (20 years) is approximately 1.68 million or about .084 million ESAL applications per year. The AASHO Test Road pavement sections underwent a measurable drop in the serviceability level after about 0.5 million applications. At the rate at which loads are being applied in 1975, the experimental pavements in sections 195-3 and 195-2 at Rock Falls will be at least 6 years old before any noticeable difference in serviceability level develops, assuming that no increase in traffic volume occurs and that the pavement sections behave similarly to those in the AASHO Test Road.

CONCLUSIONS

The objective of this research was to evaluate the behavior of plain, jointed portland cement concrete pavement by comparing it to an adjacent section of continuously reinforced pavement in the same environment. For this study, pavement behavior was defined as variations in surface condition, in riding quality, and in Present Serviceability Index between the two types of pavement with respect to changes in time and traffic. The accumulation of the necessary data to make the evaluation was begun in the fall of 1973, when the experimental pavement was opened to traffic. The CRC control section was not completed until the fall of 1975 so that the amount of data available for both pavements is limited. As the available data indicate, significant trends in pavement behavior relative to pavement riding quality and serviceability have not yet developed because no significant change in pavement surface condition that can be associated with time and traffic has occurred. On the basis of the traffic volume and with allowance for the pavement strength, changes in behavior are not expected for at least 5 or more years of service.

Although the observations and measurements taken thus far are limited, they do warrant the following conclusions:

1. The initial difference in pavement smoothness between the experimental plain, jointed pavement and the continuously reinforced pavement is attributed to the construction techniques that were employed rather than to pavement behavior.
2. Significant differences in pavement performance, which are associated with Roughness Index, cracking, patching, and traffic, are not expected for several years.
3. When joints widen from 0.035 in. to 0.04 in. (0.9 mm to 1.0 mm), aggregate interlock as a means of load transfer is lost (3). In view of the mean summer and winter temperature differentials that were observed and the winter joint openings that were measured, aggregate interlock cannot be an efficient load-transfer system across closely spaced transverse contraction joints in the Rock Falls area.
4. The dowelled load-transfer system is more effective than aggregate interlock in reducing pavement deflection at the pavement panel corners.
5. The absence of differential faulting now between the pavement test section with dowels and that without dowels should be associated more with subgrade support, pavement thickness, joint skewness, and low traffic volume rather than with aggregate interlock as a load-transfer system (4).

RECOMMENDATIONS

Within the limits of this study, the following recommendation seems to be justified: Continue the use of dowel bars as a load-transfer device in jointed pavements that become a part of either Interstate or primary highways.

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