1. Confidence levels of 95 and 99 percent are reasonable for use in designing Interstate CRCP pavements with RPS;
2. Modified AASHO performance equations used in RPS give reasonable results;
3. In lieu of traffic rates, RPS thickness designs for a 30-year analysis period are valid; and
4. This study provides partial verification of RPS, CRCP design capability.

The potential for use of RPS as a tool to design overlays on existing concrete pavements is another important aspect of RPS that should be stressed as well as the capabilities of the program to make economic comparisons of the designs. Results of this study provide one verification of RPS capabilities; however, other studies should be made to validate other areas of RPS design. The current RPS program version, RPS3, is well-documented for implementation (1) and should be used in other studies such as this one to validate and implement the program.

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Report on an Experiment for Continuously Reinforced Concrete Pavement in Walker County, Texas

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This report summarizes the findings that resulted from a 16-year study on the performance of a continuously reinforced concrete pavement placed on I-45 in Walker County, Texas. An examination of data provides numerous guidelines for design requirements and construction specifications of future projects in which this type of pavement will be used. Specifically, there were more failures for the pavement in which a lower percentage of reinforcing steel and higher curing temperatures were used. The data indicate that type 3 cement withstands higher steel stresses and that special attention should be given to concrete vibration at all times. The 7-year performance of a short section of an asphalt-concrete overlay with varying thicknesses indicates that the rate of failure and the deflection can be substantially reduced by increasing overlay thickness.

An experiment was conducted to evaluate the relative performance of 0.5 and 0.6 percent, longitudinal steel sections that were used to continuously reinforce a concrete pavement. The continuously reinforced concrete pavement (CRCP) used for this experiment was constructed during 1960 on I-45 in Walker County, Texas [Project I-45-2(3) 102; Control 675-7-4; Walker-Montgomery county line to Huntsville loop]. Since construction of the pavement, there have been numerous studies done by the Texas State Department of Highways and Public Transportation (SHPDT) and other agencies. Some of these studies have been reported in professional journals

(1, 2), other reports (4, 5, 6, 11), and SHPDT reports (3, 7, 8, 9, 10) that discuss steel stress, crack spacing, and failure studies conducted during the first 4 years of the project; failure repairs made after an age of approximately 10 years; use of asphalt overlays on the CRCP; construction and maintenance of the pavement; and various other studies conducted during the project.

Studies concerning the original surface were terminated when an asphalt-concrete overlay was placed over the entire length of the Walker County Project. However, before the overlay was placed, final surveys were conducted so that conclusions could be derived from data gathered during the 16 years of service. The objectives of this report are as follows:

1. Evaluate the relative performance of the steel percentages used to continuously reinforce the concrete pavement during the 16-year period, and
2. Consolidate the findings from all studies into one report so that the appropriate conclusions and recommendations can be formulated.

PROJECT BACKGROUND

The project begins at the Walker-Montgomery County line and proceeds northward to a point 3.21 km (2 miles) south of Huntsville. Figure 1 shows the location and general layout of the divided highway, which has two lanes of traffic in each direction. The pavement, 20.32 cm (8 in) thick and 7.32 m (24 ft) wide, was placed monolithically during the latter half of 1960 and during the spring of 1961. The subbase layer consists of open-graded sandstone, and the top layer of natural clay-sand soil 15.24 cm (6 in) thick was treated with 3 percent lime (by weight) to provide an additional layer.

Because the highway serves as a main connecting route between the Houston and Dallas metropolitan areas there is a high percentage of trucks traveling on it. Traffic counts indicate that the roadway had seven hundred and sixty, 8165-kg (18-kip) equivalent axle load (EAL) applications per day in 1960, had 4 300 000 cumulative EAL applications by 1974, and will have an estimated 5 600 000 EAL applications by 1981.

EXPERIMENTAL NATURE OF PROJECT

The 0.5 percent steel design was achieved by using number 5 bars at a center-to-center spacing of 19.05 cm (7.5 in) and the 0.6 percent steel design was achieved by using number 5 bars at a center-to-center spacing of 16.51 cm (6.5 in). In each direction, the roadway [18.19 km (11.3 miles) long] was equally divided between the two steel percentages. In addition to the steel performance study, another experimental consideration was the use of a minimum center factor of 22.308 kg/m^3 (4 sacks per yd^3), minimum and maximum flexural strengths of 3.8 kPa (550 lbf/in^2) and 4.7 kPa (675 lbf/in^2) respectively, and a specified air-entrained content of 2 to 5 percent.

As part of the development of design criteria for CRCP in Texas, the hypothesis was that a minimum concrete strength should be used to provide sufficient resistance to wheel loads and that a maximum concrete strength should be used to prevent overstressing of the steel because of the development of wide crack patterns. Thus, for several projects in the state during the 1959 to 1963 period, minimum and maximum flexural-strength specifications of 3.8 kPa to 4.7 kPa (550 to 675 lbf/in^2) respectively were used for 7 days. The specified air-entrained content was used to control strength. The two experimental steel percentages were inserted at the request of the Bureau of Public Roads (now Federal Highway Administration) to ascertain the performance variation.

During the designated period, SDHPT performed numerous studies to evaluate the performance of the pavement. Various study sections, test sections, and overlay test sections were selected from the project. The effect of flexural strength and curing temperature on average crack spacing was evaluated by studying the sections that were 122 to 183 m (400 to 600 ft) long over the duration of the project. That study gave a range for the two parameters. Two test sections were also selected for making longitudinal stress studies and a crack pattern development study for each steel percentage. The steel stress study was discontinued in 1961 and an internal summary report was prepared (1). The effect of various parameters on average crack spacing and rates of pavement failure can be found in internal and formal reports (1, 2, 3, 4, 5). The overlay test sections represented an experiment with various thicknesses of asphalt-concrete overlay to reduce the deflection incident of failure and improve riding quality. A report of these studies is given by McCullough and Monismith (5).

STEEL STRESS STUDIES

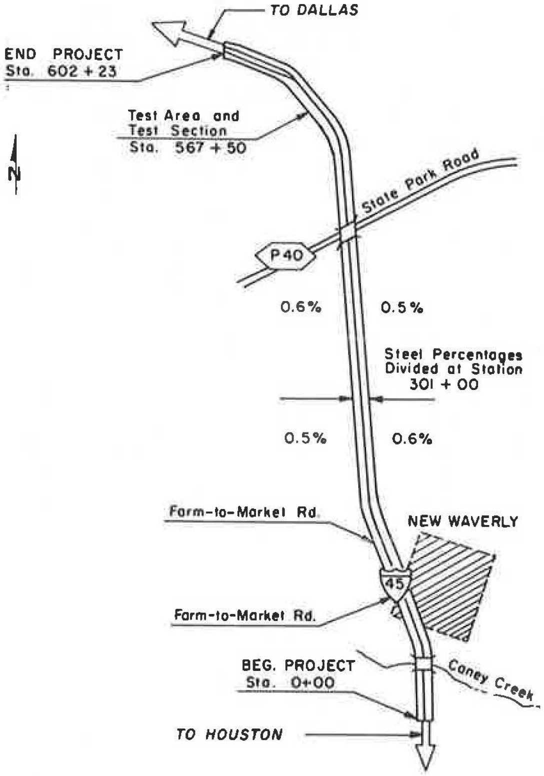
The conclusions for the study on the detailed analysis of steel stress may be found in a report (1). It was found that steel stress and concrete movement are greater at the crack than in the area between the cracks. The study indicated that the longitudinal steel stress and the concrete movement at the crack are a direct function of the slab temperature decrease and the average crack spacing and are an inverse function of the longitudinal steel percentage. These factors, which were measured by the Wagner turbindimeter test method (Tex-310D), have a significant influence on the steel stress and the concrete movement and thus should be included in any rational design procedure. In addition, it was found that the type of portland cement used had a profound influence on the steel stress at the crack. Inadvertently, during the construction, type 3 cement met the specification requirements of type 1 cement; therefore, the contractor experimented by using the type 3 cement to meet the strength specifications with that minimum cement factor.

During the early periods of concrete curing, it was found that type 3 cement produced three to four times more longitudinal steel stress than type 1 cement. The cracking in concrete with type 3 was found to be explosive in nature. The stresses and crack patterns of both pavements tended to approach each other in time, but the early differentials are of such magnitude that the use of type 3 was banned from use in CRCP in Texas. A maximum specific surface area of 2000 cm^2/gm (140907.68 in^2/lb) was included in the concrete pavement specifications to prohibit the use of type 3 cement (10). The CRCP-1 computer program developed in connection with the National Cooperative Highway Research Program (NCHRP) included these variables (11).

CRACK PATTERN OBSERVATIONS

Crack pattern observations were made at periodic intervals from the time of construction to the end of the survey. These data provided a historical development of the crack pattern over the 16-year period. Crack surveys were recorded on two test sections and eight study sections. The test sections represented the amount of pavement placed for an entire day [approximately 609.6 m (2000 ft)] for each steel percentage. These data were studied to evaluate the crack development at various points along the placement and the effect of steel percentage. The study sections, 122 to 183 m (400 to 600 ft) long, were

Figure 1. Location and layout of Walker County Project.



selected to provide information about variations in concrete strength, curing temperature, roadway direction, and steel percentage. During the final survey, seven additional sections were used to provide a large data base.

Test Sections

The effect of longitudinal steel percentage on the average crack spacing for each of the two test sections that were about 1417 m (4650 ft) long was evaluated by periodic surveys that were made since the project began. Figure 2 shows the age-crack spacing relations for these two test sections from construction to 1974. Cracking patterns had developed quickly during the first 5 months on the project. Initially, a large rapid decrease occurred in both sections as a result of curing. From about 150 days onward, only a slight decrease in the average crack spacing is seen for the next 10 to 12 years, which is mainly attributable to environmental and seasonal effects. Between 1963 and 1974, a small continued decrease was experienced in both sections because of increased traffic loading and increased rates of failures.

Study Sections

The crack patterns on the study sections follow the same trend as those on the test section; however, several significant differences were found in the 1974 data. Table 1 gives the crack-spacing data taken at different locations throughout the project in 1974. These sections were randomly selected to provide the experiment with data for ascertaining the effect of traffic direction, steel per-

Figure 2. Relation between age and average crack spacing on test sections.

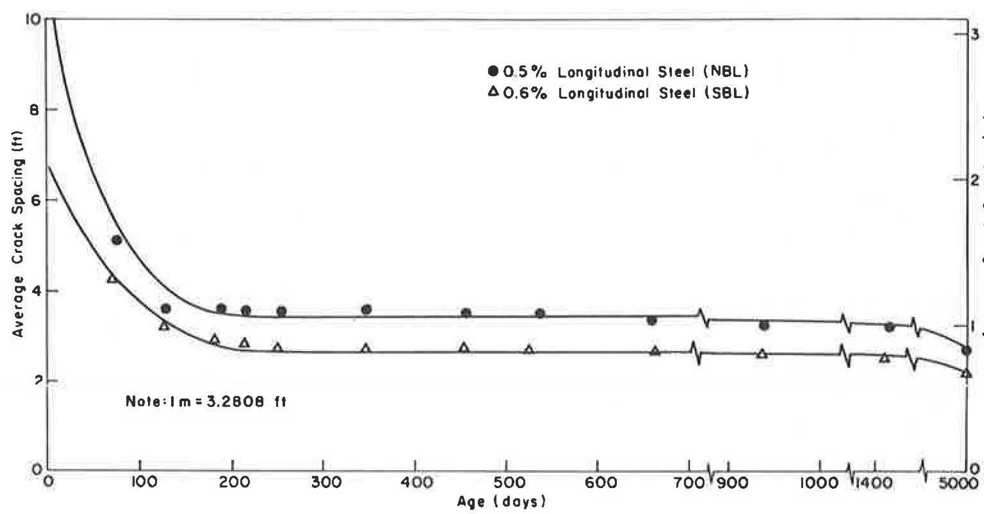


Table 1. Analysis of variance results for crack spacing data taken at various locations on the project in 1974.

Station	Section[a]	Steel Percentage	Length (m)	Number of Cracks	Crack Spacing (m)
SBL					
99 + 00 - 105 + 00	Study 6	0.5	183	242	0.76
105 + 00 - 109 + 00	—	0.5	122	160	0.76
109 + 00 - 119 + 00	—	0.5	305	306	0.99
119 + 00 - 129 + 00	—	0.5	305	288	1.06
129 + 00 - 132 + 75	—	0.5	114	97	1.18
298 + 00 - 303 + 00	Study 4	0.5, 0.6	152	236	0.65
334 + 00 - 339 + 00	Study 3	0.6	152	270	0.56
565 + 00 - 589 + 00	Test	0.6	427	825	0.63
NBL					
70 + 00 - 75 + 27	Study 7	0.6	161	195	0.82
109 + 00 - 114 + 00	—	0.6	152	139	0.88
114 + 00 - 124 + 00	—	0.6	305	396	0.77
124 + 00 - 134 + 00	—	0.6	305	405	0.75
530 + 00 - 535 + 00	Study 3	0.5	152	231	0.66
553 + 00 - 565 + 00	Test	0.5	366	504	0.72

Note: 1 m = 3.28 ft.

[a] Column indicates if the section encompasses one of the regular sections shown in Figure 3.

centage, and relative location of section on the project. The results of an analysis of variance for Table 1 are as follows (1 m = 3.28 ft):

Variable	Avg Crack Spacing (m)
Traffic	
NBL	0.75
SBL	0.76
Steel	
0.5 percent	0.82
0.6 percent	0.70
Placement	
0.5 percent steel	
North end, NBL	0.70
South end, SBL	0.89
0.6 percent steel	
North end, NBL	0.79
South end, SBL	0.61

For the above variables, only those for traffic direction were not significantly different. The average crack spacing ($\overline{X}$) did not differ appreciably between directions, i.e., northbound lanes (NBL) [0.75 m (2.47 ft)] and southbound lanes (SBL) [0.76 m (2.51 ft)]. The differences in average crack spacings between steel percentages were and have always been small but separable: 0.82 m (2.70 ft) for 0.5 percent steel and 0.70 m (2.30 ft) for 0.6 percent steel. The $\overline{X}$'s for each steel percentage in each roadway also varied as much as the percentage of steel: $\overline{X}$ = 2.91 for 0.5 percent steel on SBL and $\overline{X}$ = 2.58 for 0.6 percent steel on NBL. For both steel percentages, the crack spacing at the north end of the project was smaller than that at the south end of the project. This difference indicates that the cooler curing temperatures used for the pavement at the south end of this roadway were a controlling feature. (In general, the temperatures during concrete placement were cooler at the

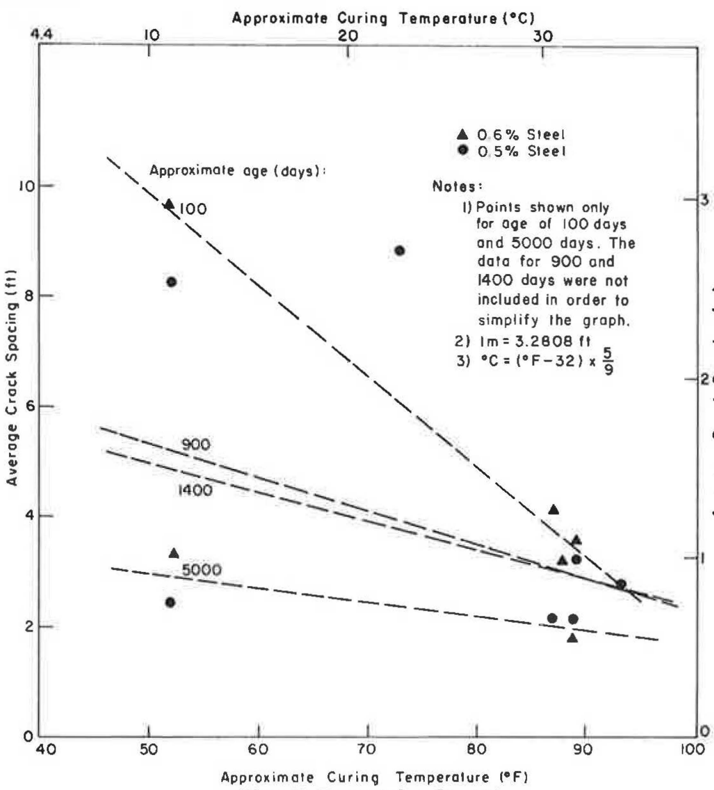
Table 2. Average crack spacing for all sections.

Survey	Date	Test Section 1	Test Section 2	Study Section 0	1	2	3	4	5	6	7
1	8/26/60	3.10	2.01	4.69	—	—	—	—	—	—	—
2	10/26/60	1.53	1.29	1.85	—	—	—	—	—	—	—
3	12/21/60	1.08	0.96	1.25	—	—	—	—	—	—	—
4	2/17/61	1.05	0.89	1.08	—	—	—	—	—	—	—
5	3/17/61	1.05	0.85	0.96	1.00	0.85	1.07	1.23	2.73	2.54	2.92
6	4/27/61	1.04	0.83	—	—	—	—	—	—	—	—
7	7/27/61	1.04	0.83	0.94	0.96	0.83	0.96	1.22	2.69	2.41	2.28
8	12/16/61	1.04	0.83	—	—	—	—	—	—	—	—
9	3/15/62	1.04	0.82	—	—	—	—	—	—	—	—
10	7/7/62	1.01	0.80	—	—	—	—	—	—	—	—
11	5/12/63	0.98	0.78	0.88	0.89	0.8	0.86	1.03	1.62	1.45	1.26
12	9/16/74	0.72	0.59	0.77	0.65	—[a]	0.56	0.65	—	0.99	0.94

Note: 1 m = 3.28 ft.

[a] Located in overlay test section.

Figure 3. Average crack spacing versus approximate curing temperature for study sections.



south end than at the north end.)

Table 2 gives the crack-spacing data for the various test sections that were observed during the life of the facility. The study sections had been selected earlier to provide a range in design factors such as steel percentage, flexural strength, and curing temperature.

Earlier studies had indicated that several factors such as relative position within a slab from a construction joint, steel percentage, average 7-d flexural strength, and air entrainment percentage affected the average crack spacing. Initially, a strong interrelation existed, but all the relations have been progressively nullified with time to the point that by 1974, no positive relation existed between average crack spacing and any of the above investigated factors, except for curing temperature (Figures 3 and 4).

These data show that slabs with the same steel percentage will have the same crack spacings over a long period of time even though the curing temperature, flexural strength, and location for the amount of pavement placed for an entire day may vary. Since attempts were made to control the maximum and minimum flexural strengths, the effect of flexural-strength variation cannot be fully evaluated on this project because the range was small. Generally, crack patterns develop at different rates during the first years; therefore, these observations should not be construed to mean that curing temperature is not an important factor for consideration in design. Thus, as previously indicated, the steel stresses and consequently the performance will vary significantly.

Figure 4. Average crack spacing versus steel percentage.

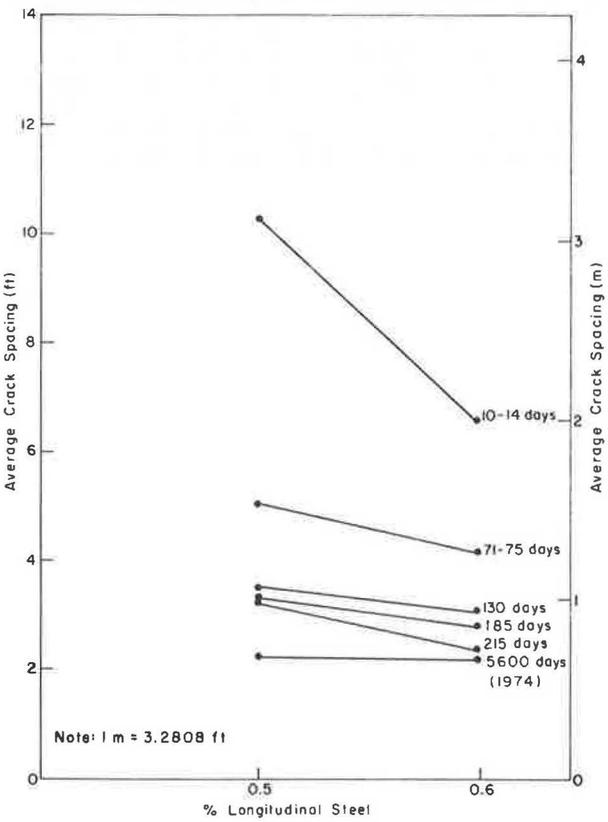
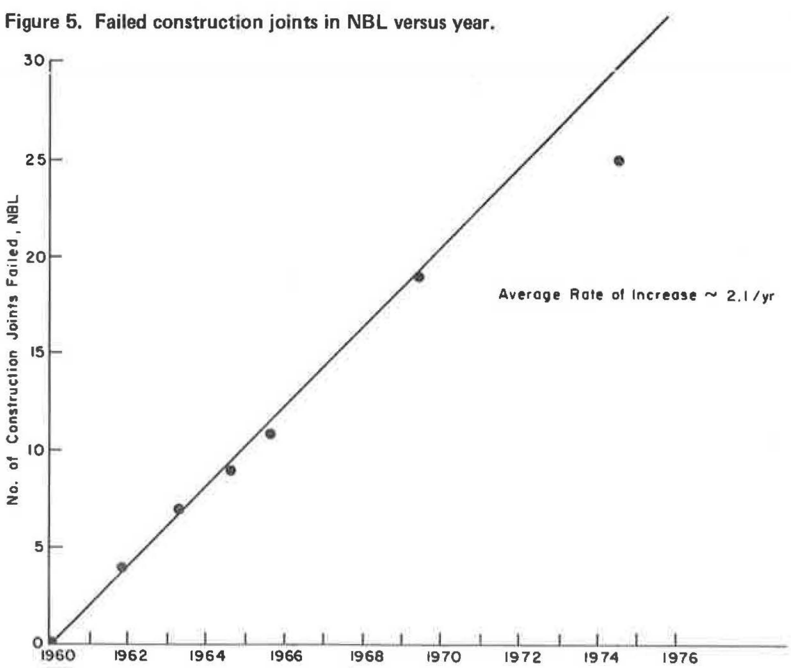


Figure 5. Failed construction joints in NBL versus year.



DEFLECTION STUDIES

Deflection studies were made on this project for three different purposes: the first two were concerned with the behavior of CRCP, and the third was concerned with the experimental overlay. In 1962, the first study investigated the surface irregularities found on the project. The second study attempted to determine the effect that steel percentage had on deflection. Shortly after the project was opened to traffic, surface irregularities were noticed in the vicinity of the construction joints at numerous locations over the length of the project. It was found that on the down placement side of the construction joint excessive deflection was occurring. According to the American Association of State Highway Officials (AASHO) deflection data, the pavement in these troubled areas was acting similar to that of a 14-cm (5.5-in) road test pavement; whereas, the satisfactory sections were deflecting similar to that of a 24.1-cm (9.5-in) road test pavement. The results of a subsurface investigation showed that the down side of the construction joint received inadequate vibration in the lower part of the slab, which caused the bottom 7.6 to 10.2 cm (3 to 4 in) of the slab to become honeycombed. As a result, the effective thickness of the slab ranged from 10.2 to 12.7 cm (4 to 5 in), and thus the data agreed with the results of the deflection study. As a result of that study, it is suggested that, for all future jobs, extra precautions should be taken in vibrating the concerte on the down side of a construction joint. Additional requirements were added to the design standards and specifications.

Studies on the effect that steel percentage had on deflection were inconclusive. Generally, there seemed to be no apparent trend that indicated that the sections with a higher percentage of steel exhibited less deflection. Thus, it was tentatively concluded that, if there was enough steel in the slab to retain the aggregate interlock, the slab would act as a continuous unit. Although followup studies were not conducted in 1974, a limited study during the life of the facility indicated no apparent change in these observations.

PERFORMANCE STUDIES

Overall, the riding qualities of the roadway on the Walker County Project were very good, especially when compared to those of the jointed concrete pavement project that is to the north and south of the Walker Project. However, as early as 1962, a large number of failures occurred in the pavement. In 35.4 km (22.6 miles) of roadway, 35 failures occurred by 1964, 109 by 1969, and over 350 by 1974. These failures occurred both at and between construction joints. The term failure is used to describe a serious disintegration of the pavement structure that includes patches, repairs, punchouts, and severe spalling. Over the years, these failures have been correlated with numerous factors related to pavement construction such as mix design, flexural strength, and curing temperature.

Construction Joint Failures

Shortly after the project opened, serious failures were found to have developed quickly at several construction joints. The repairs made on these areas verified what the deflection studies had shown in that the lower 7.6 to 10.2 cm (3 to 4 in) of pavement thickness could not be counted on to act as pavement because the concrete beneath the reinforcement mat was seriously honeycombed. The effective depth of the pavement in these areas was from 10.2 to 12.7 cm (4 to 5 in). Since hand vibrators were not required at construction joints, sufficient vibration of the bottom 7.6 to 10.2 cm (3 to 4 in) did not occur in the range of 6.1 m (20 ft) from a construction joint. In 1965, a nuclear road density logger was used to determine how widespread the honeycombing problem was in the pavement. Moderate success was achieved by using this method. It was predicted that 70 percent of the construction joints would fail because of honeycombing; however, this estimate was thought to be unrealistic at the time.

A history of construction joint failures on the project was compiled over the years. At the time of the overlay, approximately 75 percent of the construction joints in NBL had experienced failures. This percentage is similar to the percentage predicted by using the nuclear road logger. Although there was some question as to the magnitude of the amount of failures during 1965, the 1974 data indicate that the prediction is reliable. Hence, the feasibility of using such equipment to identify the problem area is reinforced. Figure 5 shows the rate of increase in failures per year and age as a linear relation on this project. However, this rate of increase must be correlated with the traffic build-up on the project. The average increase per year in construction joints from 1960 through 1974 was 2.1, i.e., every year 2.1 additional construction joints in this pavement fail because of excessively close crack spacing, punchouts, and spalling. Of the construction joints that have less than 2 d separation between placement, 82 percent (23 of 28) experienced some type of failure. Of the construction joints that have 2 d or more between placement, 57 percent (8 of 14) experienced failures. This difference is significant; however, it may be because of general construction practices rather than the steel strength properties of the concrete.

Intermediate Failures

By 1963, there was a rise in the number of failures between constructions. These failures have frequently been mentioned and studied by previous investigators. The primary reason for the failures was believed to be flash sets of the concrete during paving operations in hot weather. In 1964, a significant early trend was detected between the percentage of failures in a pavement slab versus the curing temperature of that slab (2). This same type of information was analyzed for the years 1969 and 1974 for the same sections. No definitive correlation exists for these years.

It is felt that, over the years, the differing weathering, soil support, traffic, and pavement properties had a more significant effect on failure than did that of the initial curing temperatures. However, part of the problem in analyzing the data is evidenced by the survey methods themselves and by the manner in which the surveys were made. The survey methods used were visual and photographic. Photographs cannot capture all the failures, and an experienced technician cannot perfectly describe the extent and seriousness of a certain failure. Also, a visual survey was made the first year, a photographic survey was made the second year, and no survey was made the third year. Thus, the limitations of these methods significantly affect the data analysis. Although each survey can pinpoint trends in one pavement, it cannot be viably related to the magnitude of trends found by the other survey method; therefore, a source for experimental error was introduced into this analysis.

The following is the number of failures in terms of steel percentages and roadway direction for 1964 and 1974.

Traffic Direction	Number of Failures 1964	1969
0.5 percent steel		
NBL	13	186
SBL	4	96
0.6 percent steel		
NBL	4	97
SBL	14	59

It appears from the 1969 data that substantially more failures were observed in the 0.5 percent steel sections than the 0.6 percent steel sections. Furthermore, there are substantially more failures in NBL than in SBL. Approximately 43 percent of the failures occur in the NBL and 0.5 percent combination, which is a low-steel percentage and a high-curing temperature condition, whereas, the low-temperature curing and high-steel percentage combination had only 13 percent of the failures.

Table 3 gives the percentage of the roadway experiencing failure in NBL and SBL for a range of maximum air temperatures during concrete placement. By using both the 1969 and 1974 data from the road repair survey, it is seen that substantially more failures occurred when the concrete placement temperature was in the range of 32 to 37°C (90 to 99°F). The same trend is evident in the SBL, although the percentages are not as high.

Experimental Overlay

An experimental asphalt-concrete overlay that had varying thicknesses was placed over CRCP with both steel percentages in 1969. The section overlaid was 1417.32 m (4650 ft) in both NBL and SBL. A profile of the thickness variation is shown in Figure 6, and thicknesses were 5.1, 10.2, and 15.3 cm (2, 4, and 6 in). Dynaflect readings were taken on sections both before and after overlay to determine the effect of the overlay on the load-carrying capability of the pavement (Table 4).

A significant decrease occurred in the dynaflect readings after the overlay. The dynaflect readings, if averaged for NBL and SBL and plotted against the thickness of overlay, show a strong interrelation (Figure 7). As expected, the readings significantly decrease with an increase in thickness. The same data are plotted as percentage reduction of dynaflect readings versus the thickness of overlay (Figure 8). The graph shows approximately 5 percent deflection reduction for each 25.4 mm (1 in) of asphalt-concrete pavement.

When the overlay was placed, the average crack spacing of CRCP was estimated to be 0.67 m (2.2 ft) in NBL. Since the overlay was placed, cracks developed in all thicknesses of asphalt overlay. This cracking is termed reflection cracking, i.e., cracking that is in the overlay, which forms at or near the crack in CRCP. The percentage of reflection cracking is defined as the ratio of crack spacing in CRCP in 1969 to the crack spacing in the overlay in 1974, which is then multiplied by 100. A plot of the percentage of reflection cracking versus overlay thickness (Figure 9) shows a very strong decrease in the reflection-cracking percentages, as expected. Note that for the 15.2-cm (6-in) overlay, zero reflection cracking was experienced. A design study indicated at least 6.4 cm (2.5 in) of asphalt-concrete pavement was needed to prevent reflection cracking.

Figure 6. Experimental asphalt-concrete overlay in 1969.

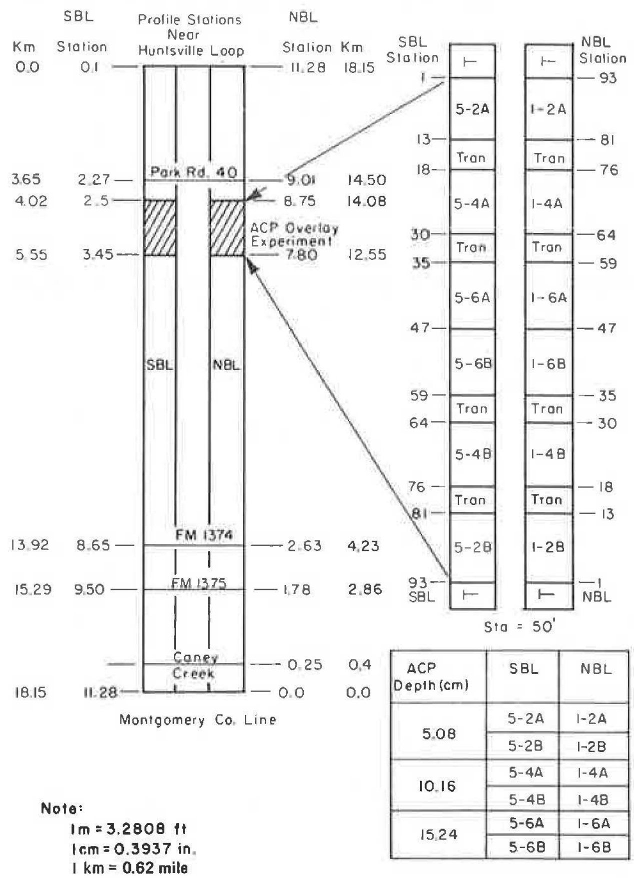


Table 3. Percentage of roadway experiencing failure during 1969 and 1974 for NBL and SBL.

Curing Temperature (°C)	1969		1974		
	NBL[a]	SBL[a]	NBL[a]	NBL[b]	SBL[b]
4.4 to 9.4	3.45	1.31	3.26	2.74	2.81
10.0 to 15.0	1.56	4.39	2.53	5.10	1.40
15.5 to 20.5	2.94	1.33	3.70	4.39	1.20
21.1 to 26.1	4.83	9.30	7.04	2.93	3.28
26.7 to 31.7	2.72	7.15	8.72	2.79	2.64
32.2 to 37.2	7.49	5.40	19.03	8.80	5.92

Note: 1°C = (°F - 32) x 5/9.
[a] From actual road repairs. [b] From photographic survey.

Table 4. Dynaflect deflection readings before and after overlay.

	Section											
	2B		4B		6B		2A		4A		6A	
Date	Avg	Std.	Avg	Std.	Avg	Std.	Avg	Std.	Avg	Std.	Avg	Std.
NBL												
Before												
6/67	0.024 485	0.000 172	0.019 355	0.000 139	0.020 980	0.000 176	0.919 685	0.000 143	0.009 626	0.000 137	0.021 209	0.000 218
9/67	0.027 407	0.000 226	0.019 482	0.000 144	0.021 742	0.000 327	0.021 209	0.000 133	0.022 834	0.000 112	0.022 225	0.000 215
11/67	0.024 587	0.000 171	0.019 202	0.000 119	0.021 412	0.000 230	0.023 749	0.000 313	0.027 889	0.000 166	0.027 991	0.000 376
1/68	0.028 118	0.000 247	0.020 244	0.000 145	0.023 261	0.000 402	0.023 393	0.000 184	0.030 353	0.000 212	0.029 235	0.000 372
After												
2/68	0.023 520	0.000 369	0.015 062	0.000 089	0.014 300	0.000 148	0.019 964	0.000 102	0.021 158	0.000 132	0.017 653	0.000 150
8/68	0.023 470	0.000 407	0.014 402	0.000 093	0.013 157	0.000 156	0.016 256	0.000 124	0.017 475	0.000 106	0.014 757	0.000 124
1/69	0.025 730	0.000 435	0.014 859	0.000 090	0.014 122	0.000 152	—[a]	—[a]	—[a]	—[a]	—[a]	—[a]
SBL												
Before												
9/67	0.020 828	0.000 076	0.017 424	0.000 099	0.016 967	0.000 160	0.016 713	0.000 066	0.022 250	0.000 134	0.021 209	0.000 089
11/67	0.025 476	0.000 103	0.021 895	0.000 115	0.020 777	0.000 178	0.021 209	0.000 089	0.029 413	0.000 246	0.023 266	0.000 226
1/68	0.019 380	0.000 074	0.015 951	0.000 075	0.018 720	0.000 138	0.020 675	0.000 089	0.027 991	0.000 215	0.020 244	0.000 180
After												
2/68	0.022 123	0.000 115	0.017 043	0.000 187	0.013 513	0.000 121	0.018 644	0.000 068	0.019 126	0.000 090	0.016 815	0.000 142
8/68	0.017 272	0.000 092	0.014 122	0.000 161	0.012 243	0.000 123	0.015 215	0.000 046	0.016 510	0.000 056	0.013 767	0.000 114
1/69	0.023 546	0.000 158	0.015 799	0.000 126	0.013 030	0.000 112	0.018 745	0.000 056	0.019 914	0.000 095	0.016 713	0.000 157

Note: All average and standard dynaflect readings are in millimeters (1 mm = 0.03937 in).
[a] Indicates no reading taken.

Figure 7. Dynaflect deflection readings after overlay sections versus overlay thickness.

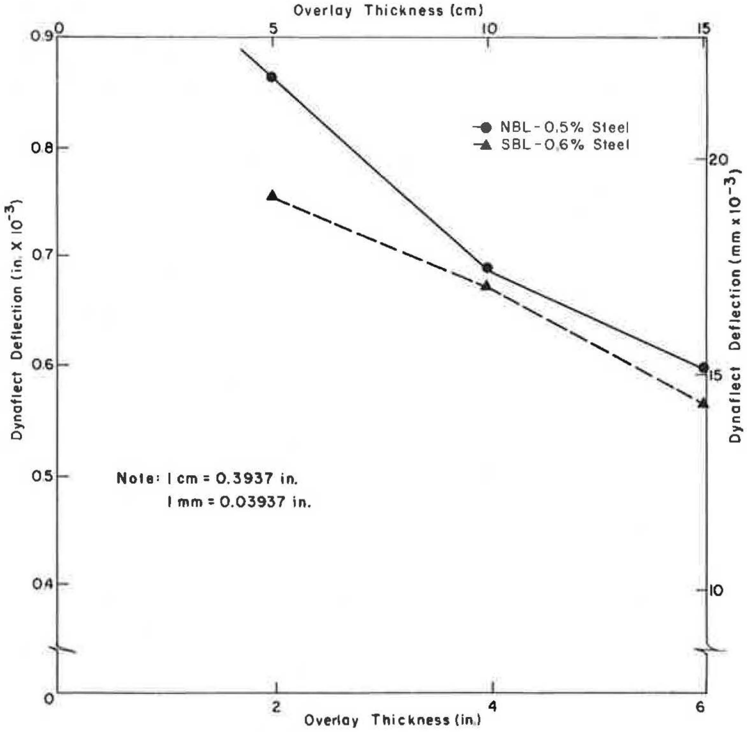


Figure 8. Decrease in deflection readings versus overlay thickness.

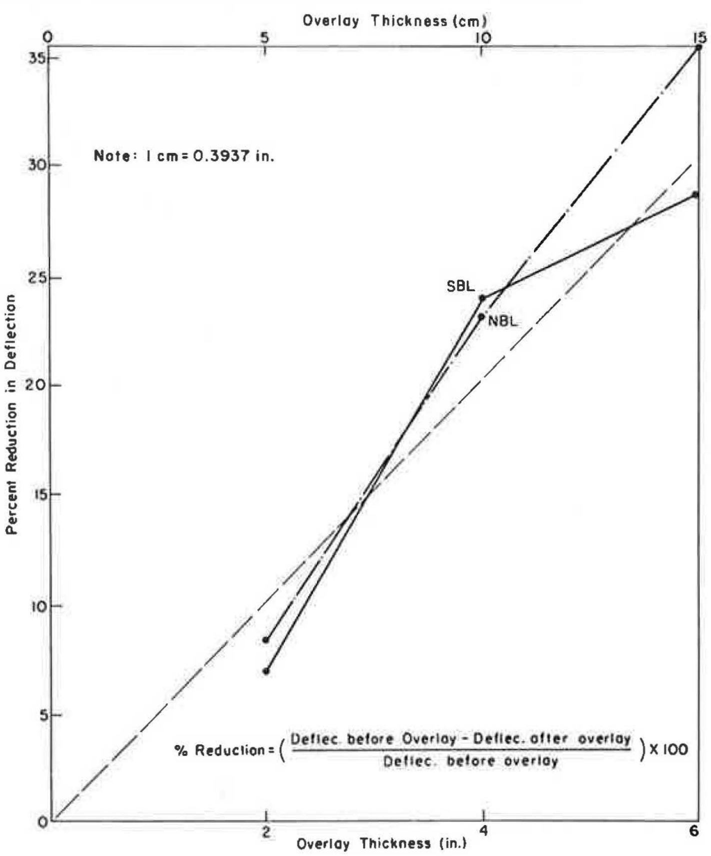
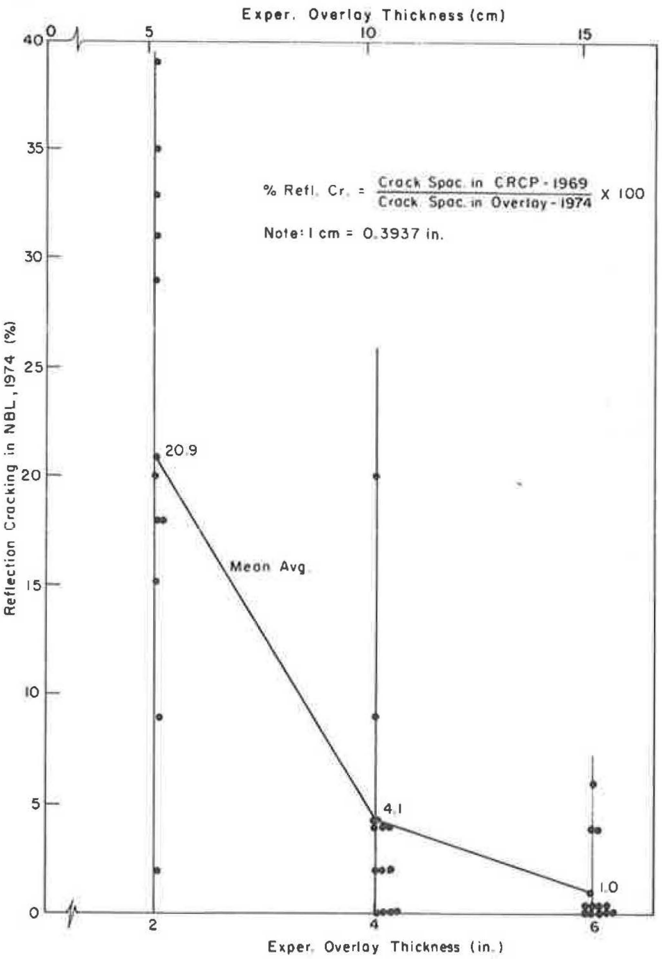


Figure 9. Reflection cracking versus overlay thickness in NBL for 1974.



Thus, this procedure should be modified in light of these data.

1974 Overlay

The Walker County CRCP experiment ended in 1974 when the length of the roadway, both NBL and SBL, was overlayed with asphalt concrete. Currently, its behavior is being studied as a flexible pavement over a CRCP. CRCP was placed in as good a condition as possible before overlay, i.e., all patches were repaired or replaced with concrete and most of the failures were remedied.

DISCUSSION OF RESULTS

The 16-year performance history of the Walker County experimental project provides an excellent insight into the construction and maintenance guidelines for CRCP. Even though numerous failures were present, the riding quality of the pavement always remained high. Thus, the importance of the visible distress manifestations in the pavement was emphasized in that they can be used by an engineer to rate the performance of a pavement. At the time of overlay, the average present serviceability index (PSI) was 3.0, which is above the generally accepted value of 2.5. Failures are visibly apparent, i.e., they can be seen as one rides over the pavement. Thus, even though the PSI is high, the visibility of the failures has an effect on the pavement-rating performance.

During the preparation of the plans and specifications for the project, a critical oversight was made by not requiring concrete vibration. The specifications for concrete pavement were adopted without including a vibration requirement; hence, the contractor was not required to adequately vibrate the concrete. This lack of vibration resulted in numerous problems that showed up during the 16 years of pavement performance before the asphalt overlay.

The first area experiencing problems was the concrete on the down placement side of a transverse construction joint (morning placement). In this area, the equipment used for concrete placement did not adequately vibrate the concrete; therefore, the area immediately below the steel became honeycombed, and this resulted in failures. The use of fine-grind cement resulted in high stresses, thus, the effect of the steel percentage showed up clearly during the performance period. There were substantially more failures in the 0.5 percent sections than in the 0.6 percent sections.

Another problem evident on the project was that concrete placed on days with high atmospheric temperatures experienced more failures. Substantially more failures were found on slabs placed when the temperature was 32.2°C (90°F) or above than at lower temperatures.

In examining the performance of the project from an overall viewpoint, it is apparent that steel stress, average crack spacing, and pavement performance were affected by the percentage of longitudinal steel, the cement type, the change in temperature from the curing temperature, and the construction techniques on the project. Thus, any design procedures for CRCP should reflect these factors. The computer program recently developed in connection with the NCHRP accounts for many of these factors in the prediction of stresses, crack width, and crack spacing for a project (11). In the past, one standard design has been used regardless of the location in the state, type of subbase used, or time of placement. It is evident from the findings of this study and the NCHRP study, that all of the abovementioned factors should be taken into account when designing a project. Hence, the slabs should be designed for a range of conditions, and, then, use a specific condition on a project basis, rather than using one pavement standard, as has been done in the past.

CONCLUSIONS

1. The use of a type 3 cement with CRCP results in cracking of an explosive nature that produces a high initial stress level in the steel, possibly even overstressing the steel.
2. Results at the end of 16 years indicated that the percentage of longitudinal steel was the only factor that influenced crack spacing. However, it should be kept in mind that steel stresses during the entire period were influenced by the other factors and thus they are important in design.
3. A study of the failures on the project indicated more failures were experienced with 0.5 percent longitudinal steel than with 0.6 percent longitudinal steel, and more failures were experienced with high-curing temperatures than with low-curing temperatures. The maximum failures were observed in areas where 0.5 percent longitudinal steel and high-curing temperatures were used.
4. Good vibration during construction is necessary for satisfactory pavement performance.
5. Deflection measurements before and after an asphalt-concrete overlay indicated that the deflection reduction was approximately 5 percent for each 25.4 mm (1 in) of overlay.

RECOMMENDATIONS

1. The maximum specific surface area requirement currently used in the specifications for CRCP should be retained. The performance over a 16-year period indicates the necessity for prohibiting fine-grind cement on a large-scale basis.

2. Consideration should be given to revising the specifications to provide closer control of concrete during hot weather placement.

3. Measuring techniques for deflection and the use of a nuclear road logger should be considered on future projects to help locate problem areas, especially for those in which there is concrete honeycombing or low density.

4. The CRCP for a given project should be designed specifically by taking into account the variables enumerated in the conclusions. The CRCP-1 computer program currently available to SDHPT can be used to design the steel and concrete for a specific project by taking into account the factors that are known to influence the pavement performance.

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Effectiveness of Pressure-Relief Joints in Reinforced Concrete Pavements

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This paper discusses the effectiveness of a 100-mm (4-in) wide compressible material that was installed at 305-m (1000-ft) intervals in a jointed, reinforced concrete pavement to reduce pavement blowups. The studies were made on an Interstate highway that carries some 30 000 vehicles/d, which includes approximately 7000 trucks and buses. This paper compares the behavior of the pavement both before and after the installation of the pressure-relief joints. Brief discussions of the factors that indicate the need for such joints, the problems associated with their use, and the potential for their use under overlays are included.

The performance of jointed concrete pavements in some areas of Virginia has been seriously impaired by the infiltration of incompressible materials into the joints, which results in blowups. This infiltration can come from below the pavement because of the slab-pumping action related to water trapped below the pavement structure, or it can come from above the pavement because of poorly sealed transverse joints. Water is entrapped when the densely graded subbase materials prohibit drainage through the shoulder (1). Transverse joints are poorly sealed when the long slabs and narrow joints, which have seasonal hydrothermal movements, are in excess of the capabilities of the sealing materials (2). The causes and mechanism of blowups in the state have been discussed in a report by Tyson and McGhee (3).

Corrective action to overcome pumping and blowup problems in Virginia has not been totally successful. Pavement-edge drains are effective in removing en-