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16. Abstract

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by

B. Frank McCullough
Thomas P. Chesney

Research Report Number 177-6

Development and Implementation of the Design, Construction and Rehabilitation of Rigid Pavements

Research Project 3-8-75-177

conducted for

Texas
State Department of Highways and Public Transportation

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by the

CENTER FOR HIGHWAY RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

April 1976

PREFACE

This report represents a summary of data collection and analysis over a 16-year period. During that period, numerous findings presented herein resulted in changes in specifications and design standards. These data should also be valuable for shaping guidelines for future construction.

The success of the study can be attributed to the efforts of many individuals over the years. Although it is impossible to list all participants, special thanks are extended to the following past and present employees of the SDHPT: M. D. Shelby (former Research Engineer, D-8), Conrad Derdeyn (D-8), William Ledbetter (D-8), W. R. Hudson (D-8), Joe Hanover (District Engineer, District 17), Jerry Nemec (Resident Engineer, District 17), Robert Long (Laboratory Engineer, District 17), and D. D. Williamson (Design Engineer, District 17).

Thanks are also extended to the Center for Highway Research staff who assisted in the project, with a special thanks to James Long and Pieter Strauss.

LIST OF REPORTS

Report No. 177-1, "Drying Shrinkage and Temperature Drop Stresses in Jointed Reinforced Concrete Pavement," by Felipe R. Vallejo, B. Frank McCullough, and W. Ronald Hudson, describes the development of a computerized system capable of analysis and design of a concrete pavement slab for drying shrinkage and temperature drop.

Report No. 177-2, "A Sensitivity Analysis of Continuously Reinforced Concrete Pavement Model CRCP-1 for Highways," by Chypin Chiang, B. Frank McCullough, and W. Ronald Hudson, describes the overall importance of this model and the relative importance of the input variables of the model and makes recommendations for efficient use of the computer program.

Report No. 177-3, "A Study of the Performance of the Mays Ride Meter," by Yi Chin Hu, Hugh J. Williamson, B. Frank McCullough, and W. Ronald Hudson, discusses the accuracy of measurements made by the Mays Ride Meter and their relationship to roughness measurements made with the Surface Dynamics Profilometer.

Report No. 177-4, "Laboratory Study of the Effect of Non-Uniform Foundation Support on CRC Pavements," by Enrique Jimenez, W. Ronald Hudson, and B. Frank McCullough, describes the laboratory tests of CRC slab models with voids beneath them. Deflection, crack width, load transfer, spalling, and cracking are considered. Also used is the SLAB 49 computer program that models the CRC laboratory slab as a theoretical approach. The physical laboratory results and the theoretical solutions are compared and analyzed and the accuracy is determined.

Report No. 177-5, "A Comparison of Two Inertial Reference Profilometers Used to Evaluate Airfield and Highway Pavements," by Chris Edward Doepke, B. Frank McCullough, and W. Ronald Hudson, describes a United States Air Force owned profilometer developed for measuring airfield runway roughness and compares it with the Surface Dynamics Profilometer, using plotted profiles and mean roughness amplitude data from each profilometer.

Report No. 177-6, "Sixteenth Year Progress Report on Experimental Continuously Reinforced Concrete Pavement in Walker County," by Thomas P. Chesney and B. Frank McCullough, is a summary of data collection and analysis over a 16-year period. During that period, numerous findings resulted in changes in specifications and design standards. These data will be valuable for shaping guidelines for future construction.

ABSTRACT

This report summarizes the findings of a 16-year study on an experimental CRCP placed on IH 45 in Walker County, Texas. An examination of data provides numerous guidelines for the design requirements and construction specifications of future projects where CRCP will be used. Specifically, substantially more failures were found with the lower percentage of reinforcing steel and higher curing temperatures. The data shows Type 3 Cement gives higher steel stresses and that special attention should be given at all times to concrete vibration. The seven-year performance of a short section of a variable thickness asphalt-concrete overlay shows that the rate of failure and the deflection can be substantially reduced with an increase in overlay thickness.

KEY WORDS: Pavements, concrete pavement, continuously reinforced concrete pavement, performance, deflection, asphalt concrete overlay, reflection cracking, steel stress, average crack spacing, evaluation.

IMPLEMENTATION STATEMENT

Even though numerous problems were experienced on the Walker County Experimental Project, substantial information was gained that will be of use in future design and construction. Following are suggestions for implementation based on this study:

(1) The CRC pavement design details should be revised to permit selection of given reinforcement based on the specifics of a project, such as subbase type, area of state, and season of placement.

(2) Item 366, "Concrete Pavement (Continuously Reinforced)," should be revised to provide closer control of the concrete during "hot weather" placement.

(3) The performance of the asphalt-concrete overlay should be closely monitored in the future since substantial background data are available for the original CRCP surface.

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INTRODUCTION

During 1960, an 8-inch experimental countinuously reinforced concrete pavement was constructed in Walker County on IH 45.* The experimental nature of the project was to evaluate the relative performance of 0.5 percent and 0.6 percent longitudinal steel. During the past 16 years, numerous studies by personnel of the State Department of Highways and Public Transportation (SDHPT) and other agencies have been performed on the reports. Some of these studies have been reported in technical journals and as external reports while others have been reported in the form of internal reports for the SDHPT. References 1 and 2 outline steel stress, crack spacing, and failure studies conducted during the first four years of the project. References 3 and 4 outline failure repairs made after an age of approximately ten years. Reference 5 reports a study made using asphalt overlays on the CRC pavement. References 6 and 7 are internal reports by District 17 personnel on construction and maintenance of the pavement. References 8 through 27 are internal SDHPT reports pertaining to various studies on the project. References 28 through 31 pertain to studies conducted by various agencies that included the Walker County Project as a part of their overall studies.

Recently, the Walker County Project received an asphalt-concrete overlay over its entire length, and, thus, the studies on the project of the original surface are terminated at this point. Prior to the overlay construction, final surveys were conducted to permit conclusions to be derived after 16 years of service.

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*Project Number I-45-2(3) 102; Control Number 675-7-4; Project limits from Walker-Montgomery County Line to Huntsville Loop.

Objective of the Report

The objectives of this report are to

(1) evaluate the performance of the experimental variable percent steel on pavement performance after sixteen years of traffic and

(2) consolidate the findings from all the studies into one report and formulate the appropriate conclusions and recommendations.

Project Background

The project begins at the Walker-Montgomery county line and proceeds northward to a point two 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. A typical cross section is presented in Fig 2. The pavement consists of an 8-inch slab, 24 feet wide, placed monolithically during the latter half of 1960 and the spring of 1961. The subbase layer is an open graded sandstone, and the top 6 inches, of natural clay-sand soil, was treated with 3 percent lime (by weight) to provide an additional layer.

Since the highway serves as a main connecting route between the Houston and Dallas metropolitan areas, it has a high percentage of trucks. Traffic counts indicate the roadway had 760 equivalent 18-kip axle load applications per day in 1960, had 4.3×10^6 cummulative applications by 1974, and will have an estimated 5.6×10^6 applications by 1981.

Experimental Nature of the Project

The 0.5 percent steel design was achieved using No. 5 bars at 7-1/2-inch center-to-center spacing and the 0.6 percent steel design was arrived at using No. 5 bars at 6-1/2-inch centers. Each directional roadway, 11.3 miles long, was divided equally between the two steel percentages. In addition to the steel performance study, another experimental consideration was the use of a minimum cement factor of four sacks per cubic yard, a minimum and maximum flexural strength of 550 psi and 675 psi, respectively, and a specified entrained air content of 2 to 5 percent.

As a part of the development of design criteria for CRCP in Texas, the hypothesis that a minimum concrete strength should be used to provide sufficient resistance to wheel loads and a maximum concrete strength in order

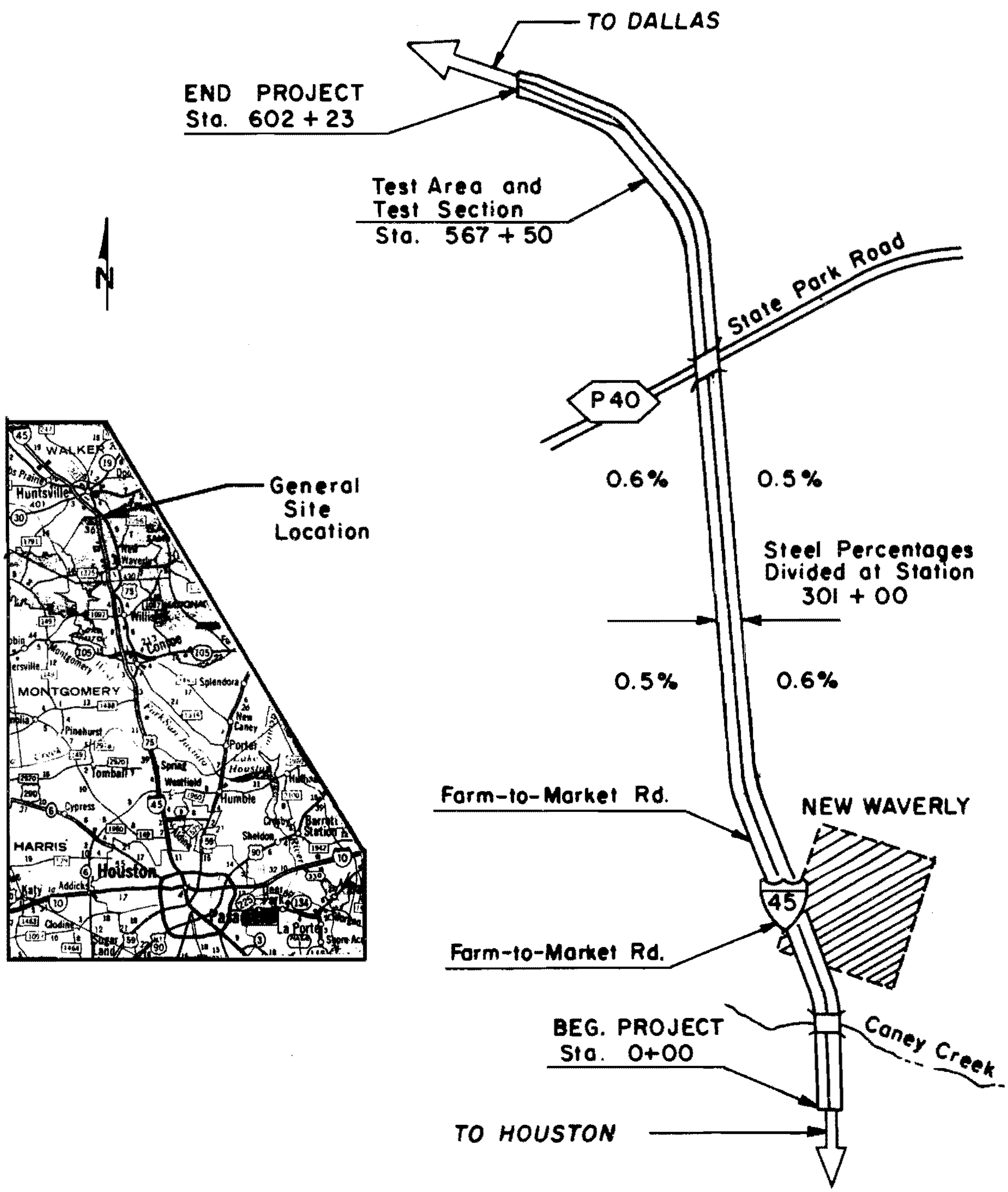

Fig 1. Location and layout of Walker County Project.

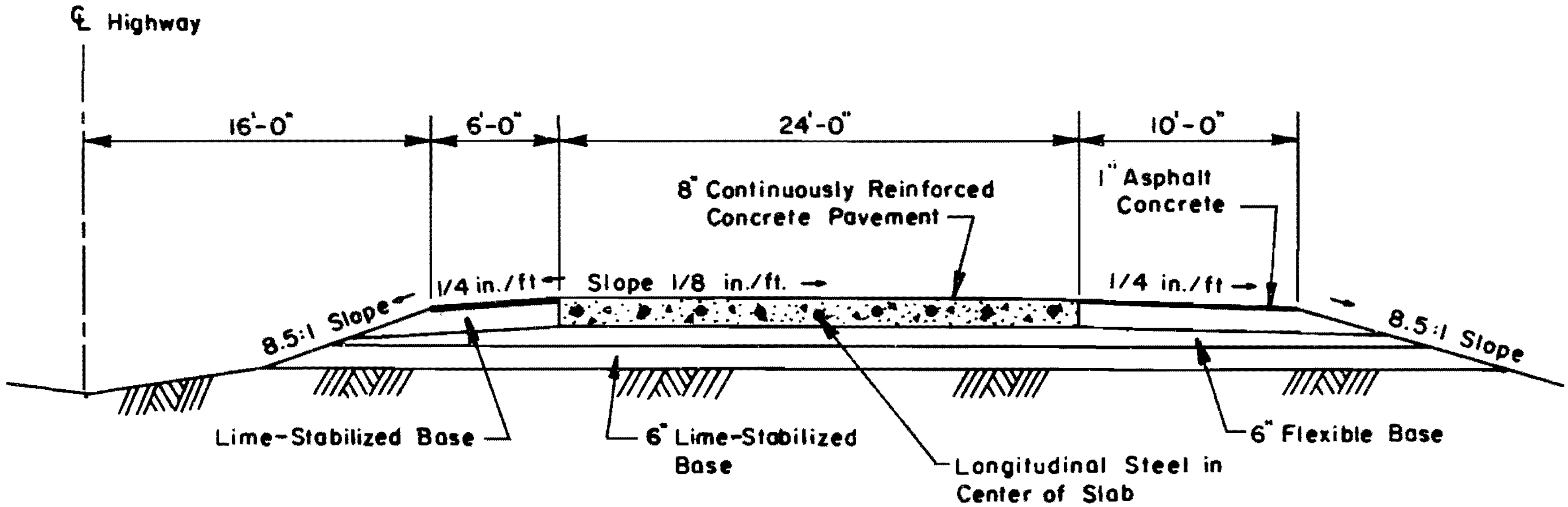

Fig 2. Typical half section for Walker County Project.

to prevent overstressing of the steel due to wide crack patterns had been developed. Thus, on several projects in the state during the period from 1959 to 1963 a minimum and maximum flexural strength specification was used (550 to 675 psi for seven days). The specified entrained air was for the purpose of controlling the strength. The two experimental steel percentages were inserted at the request of the Bureau of Public Roads (now the Federal Highway Administration, FHWA) to ascertain the performance variation.

The SDHPT performed numerous studies during the years to evaluate the pavement performance. Figure 3 shows the various study sections, test sections, and overlay test sections selected on the project. In order to evaluate the effect of flexural strength and curing temperature on average crack spacing, study sections 400 to 600 feet long were selected over the length of the project to give a range in these two parameters. Two test sections were also selected for making longitudinal stress studies and a crack pattern development study for each percentage of steel. The steel stress study was discontinued in 1961 and a summary report can be found in Ref 1. The effect of various parameters on average crack spacing and rates of pavement failure can be found in Refs 8 through 27. 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. These studies were reported in Ref 25.

SUMMARY OF FINDINGS

In this section, the findings from the various studies and final survey are summarized. These studies are divided into field stress studies, crack pattern observations, deflection studies, performance studies, and experimental overlays. For a more detailed explanation of the findings, the reader should refer to the various references given.

Steel Stress Studies

A report of the study of the detailed analysis of steel stress and its conclusions may be found in Highway Research Record No. 5 (Ref 21). It was found that steel stress and concrete movement are greater at the crack than in the area between cracks. This study indicated that the longitudinal

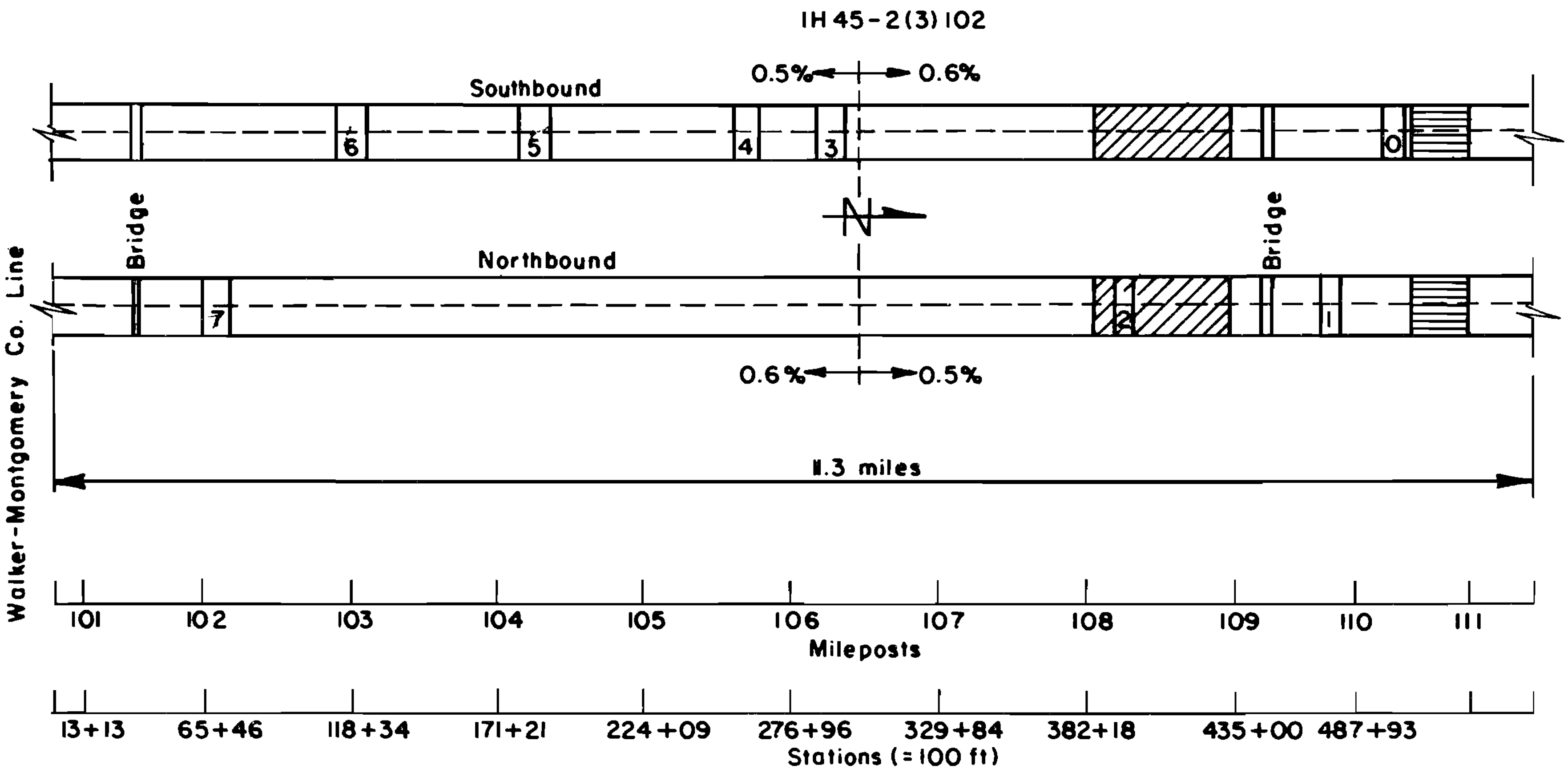

Fig 3. Experimental layout of the project and sections used in study of pavement behavior.

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 have a significant influence on the steel stress and the concrete movement, and, thus, these parameters should be included in any rational design procedure.** In addition, it was found that the type of portland cement had a profound influence on the steel stress at the crack. Inadvertently, during the construction operation both Types 1 and 3 cements were used. At the time of construction, Type 3 cement met the specification requirements of Type 1 cement, and, thus, the contractor experimented with the use of Type 3 to attempt to meet the strength specifications with a minimum cement factor.

During the early periods of concrete curing, it was found that Type 3 cement produced three to four times as much longitudinal steel stress as Type 1 cement. The cracking in concrete with Type 3 was found to be of an explosive nature. The stresses and crack patterns of both pavements tend to approach each other in time, but the early differentials are of such a magnitude that the use of Type 3 was banned from use on continuously reinforced concrete pavement (CRCP) in Texas. A maximum specific surface area of 2000 cm^2/gm*** was included in the concrete pavement specifications to prohibit the use of Type 3 cement (Ref 32).

Crack Pattern Observations

Crack pattern observations were made at periodic intervals from the time of construction to the terminal 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 as shown in Fig 3. The test sections represented an entire day's placement (approximately 2000 feet each) for each steel percentage. These data were studied to evaluate the crack development at various points along the placement and the effect

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** Measured by the Wagner Turbidimeter Test Method Tex-310D.

*** The CRCP-1 computer program developed in connection with NCHRP 1-15 included these variables (Ref 33).

of steel percentage. The study sections were 400 to 600-foot sections selected to give variations in concrete strength, curing temperature, roadway direction, and percent steel. During the terminal survey, seven additional sections were used to provide a larger data base.

Test Sections. To evaluate the effect of percent longitudinal steel on average crack spacing, two test sections, each about 2000 feet in length, were selected and periodic surveys have been made since the beginning of the project. Figure 4 shows the age-crack spacing relationships 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 due to curing. From about 150 days onward, only a slight decrease in the average crack spacing is seen for the next 10 to 12 years, mainly attributable to environmental and seasonal effects. Between 1963 and 1974, a small continued decrease was experienced in both sections due to increased traffic loading and increased rates of failures.

Figures A1.1 through A1.4 in Appendix 1 are crack pattern diagrams for each of the two sections. Several of the wide spacings on the diagram are misleading since these are repair areas and, hence, the spacing appears to be greater than normal. During the first few years after construction, the average crack spacing in both sections increased in the direction of concrete placement. The minimum spacing was present in the areas placed early in the morning, whereas those placed late in the afternoon had a wider crack spacing. The data from the last crack survey in 1974 indicated this trend had disappeared. Figure A1.5 in Appendix 1 is a cumulative frequency of crack spacing diagram for the southbound lane. The two lines represent the distribution for the first and last halves of the project. Note that the two lines are for all practical purposes overlapping; hence, the crack spacing distribution has equalized with time. Figure A1.6 shows the same trend for the test section in the northbound lane.

Study Sections. The crack patterns on the study sections follow the same trend as on the test sections, but several significant differences should be pointed out in the 1974 data. Table 1 presents crack spacing data taken at different locations throughout the project in 1974. These sections were randomly selected in order to provide an experiment to ascertain the effect of traffic direction, percent steel, and relative location on the project.

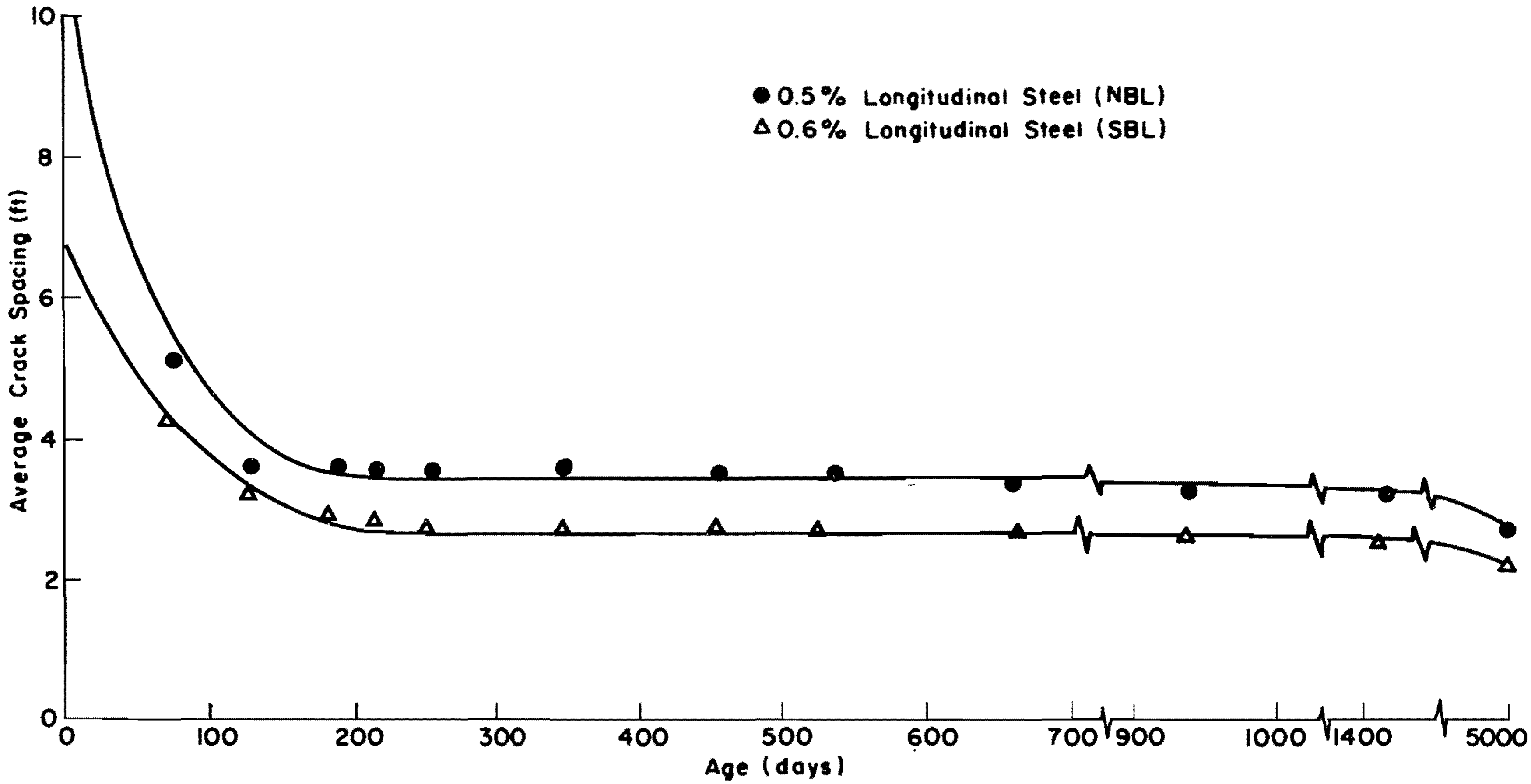


Fig 4. Relationship between age and the average crack spacing on the test sections.

TABLE 1. ANALYSIS OF VARIANCE RESULTS OF CRACK SPACING DATA TAKEN AT VARIOUS LOCATIONS ON THE PROJECT IN 1974.

Stations	Roadway Direction	Percent Steel	Section*	Length (feet)	Number of Cracks	Crack Spacing (feet)
99 + 00 - 105 + 00	SBL	0.5	Study-6	600	242	2.48
105 + 00 - 109 + 00	SBL	0.5	-	400	160	2.50
109 + 00 - 119 + 00	SBL	0.5	-	1000	306	3.26
119 + 00 - 129 + 00	SBL	0.5	-	1000	288	3.47
129 + 00 - 132 + 75	SBL	0.5	-	375	97	3.86
298 + 00 - 303 + 00	SBL	0.5/0.6	Study-4	500	236	2.13
334 + 00 - 339 + 00	SBL	0.6	Study-3	500	270	1.85
565 + 00 - 589 + 00	SBL	0.6	Test	1400	825	2.07
70 + 00 - 75 + 27	NBL	0.6	Study-7	527	195	2.70
109 + 00 - 114 + 00	NBL	0.6	-	500	139	2.88
114 + 00 - 124 + 00	NBL	0.6	-	1000	396	2.52
124 + 00 - 134 + 00	NBL	0.6	-	1000	405	2.46
530 + 00 - 535 + 00	NBL	0.5	Study-3	500	231	2.16
553 + 00 - 565 + 00	NBL	0.5	Test	1200	504	2.38

*Column indicates if the section encompasses one of the regular sections shown in Fig 3.

Variable Studied	Data Source	Average Crack Spacing (feet)	Comments
Traffic Direction	NBL SBL	2.47 2.51	not significantly different
Steel Percentage	0.5 percent 0.6 percent	2.70 2.30	significantly different
Season of Placement	(North End) (NBL, 0.5 percent) (South End) (SBL, 0.5 percent)	2.31 2.91	significantly different
Season of Placement	(North End) (NBL, 0.6 percent) (South End) (SBL, 0.6 percent)	2.58 2.01	significantly different

On the bottom part of the table, results of an analysis of variance are shown. The average crack spacing, $\overline{X}$, did not differ appreciably between directions; i.e., Northbound Lanes (NBL), 2.47 feet, and Southbound Lanes (SBL) 2.51 feet. The differences in average crack spacings between percentages of steel were and have always been small, yet separable; 2.70 feet for 0.5-percent steel and 2.30 feet for 0.6 percent steel. The $\overline{X}$'s for each percentage steel in each roadway also varied as much as the percent of steel; $\overline{X} = 2.91$ in 0.5 percent (SBL), $\overline{X} = 2.58$ in 0.6 percent (NBL). The north end of the project had a smaller crack spacing than the south end for both steel percentages. This indicates the cooler curing temperatures of the pavement in the south end of this roadway were a controlling feature. (In general, the temperatures during concrete placement were cooler at the south end than the north end.)

Table 2 contains 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 percent steel, flexural strength, and curing temperature.

Earlier studies had indicated that several factors, such as relative position within a slab from a construction joint, percent longitudinal steel, average seven-day flexural strength, and percent air entrainment, affected the average crack spacing. Initially, a strong interrelationship existed, but all of the relationships have been progressively nullified with time to the point that, by 1974, no positive relationship existed between average crack spacing and any of the above investigated factors, except curing temperature, as may be seen in Figs 5 and 6.

Summary. 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 in the day's placement may vary. The effect of flexural strength variation cannot be fully evaluated on this project, since attempts were made to control the maximum and minimum flexural strengths and, hence, the range was small.

These observations should not be construed to mean that curing temperature is not an important factor for consideration in design since the crack pattern will develop differently during the first years. Thus, as indicated in previous sections the steel stresses and, consequently, the performance will vary significantly.

TABLE 2. AVERAGE CRACK SPACING FOR ALL SECTIONS (Feet).

	Survey Number and Date of Survey											
	1	2	3	4	5	6	7	8	9	10	11	12
	8/26/60	10/26/60	12/21/60	2/17/61	3/17/61	4/27/61	6/27/61	12/16/61	3/15/62	7/7/62	5/12/63	9/16/74
Study Section												
0	15.40	6.06	4.12	3.54	3.15	-	3.10	-	-	-	2.90	2.52
1	-	-	-	-	3.29	-	3.16	-	-	-	2.91	2.13
2	-	-	-	-	2.78	-	2.74	-	-	-	2.72	*
3	-	-	-	-	3.50	-	3.16	-	-	-	2.84	1.85
4	-	-	-	-	4.03	-	4.00	-	-	-	3.38	2.12
5	-	-	-	-	8.96	-	8.83	-	-	-	5.31	-
6	-	-	-	-	8.33	-	7.90	-	-	-	4.76	3.24
7	-	-	-	-	9.6	-	7.50	-	-	-	4.14	3.08
Test Section												
1	10.16	5.03	3.54	3.45	3.45	3.43	3.43	3.41	3.41	3.31	3.21	2.36
2	6.59	4.24	3.14	2.92	2.80	2.73	2.73	2.73	2.70	2.63	2.58	1.93

*Located in overlay test section.

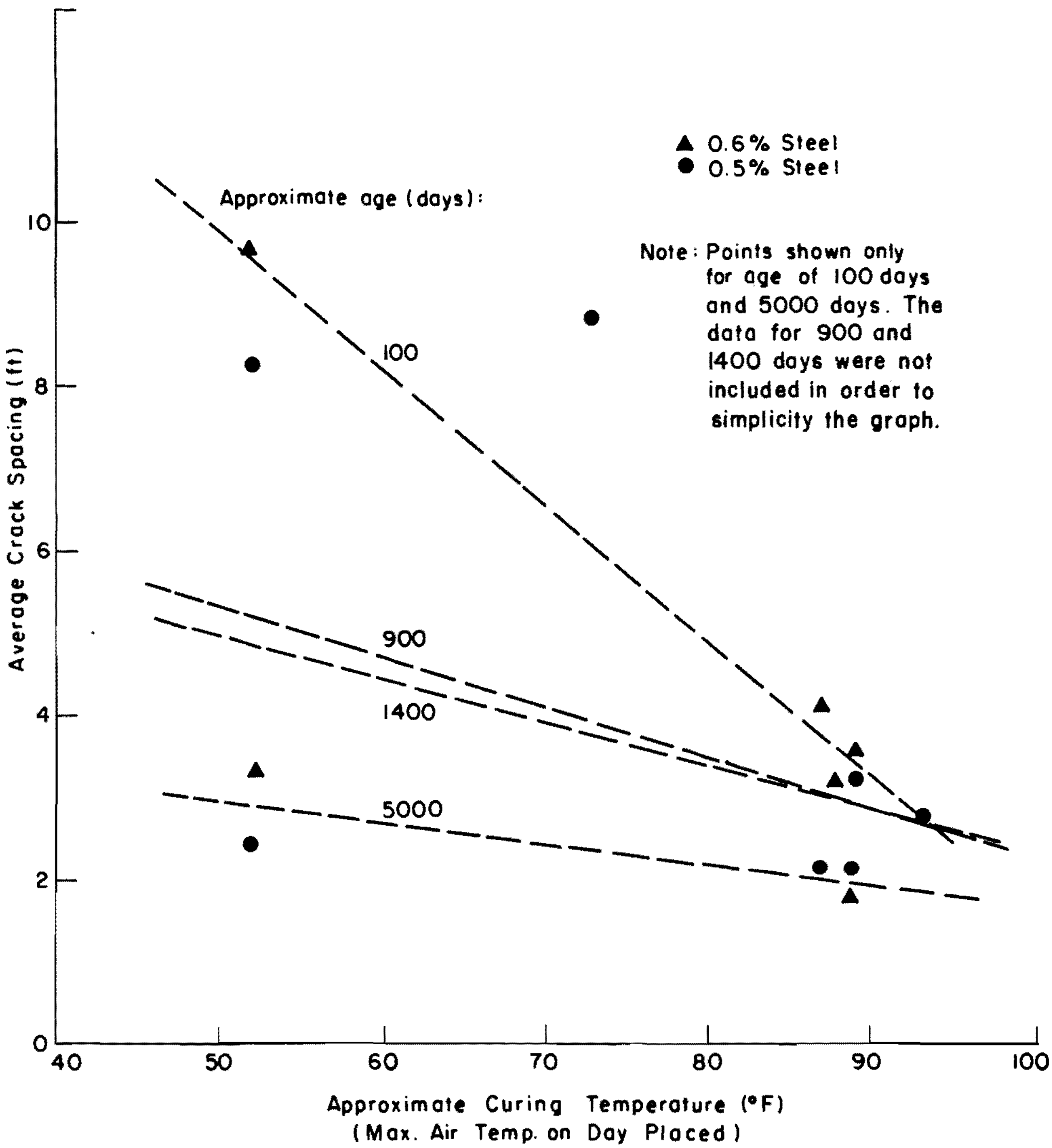


Fig 5. Average crack spacing versus approximate curing temperature for study sections.

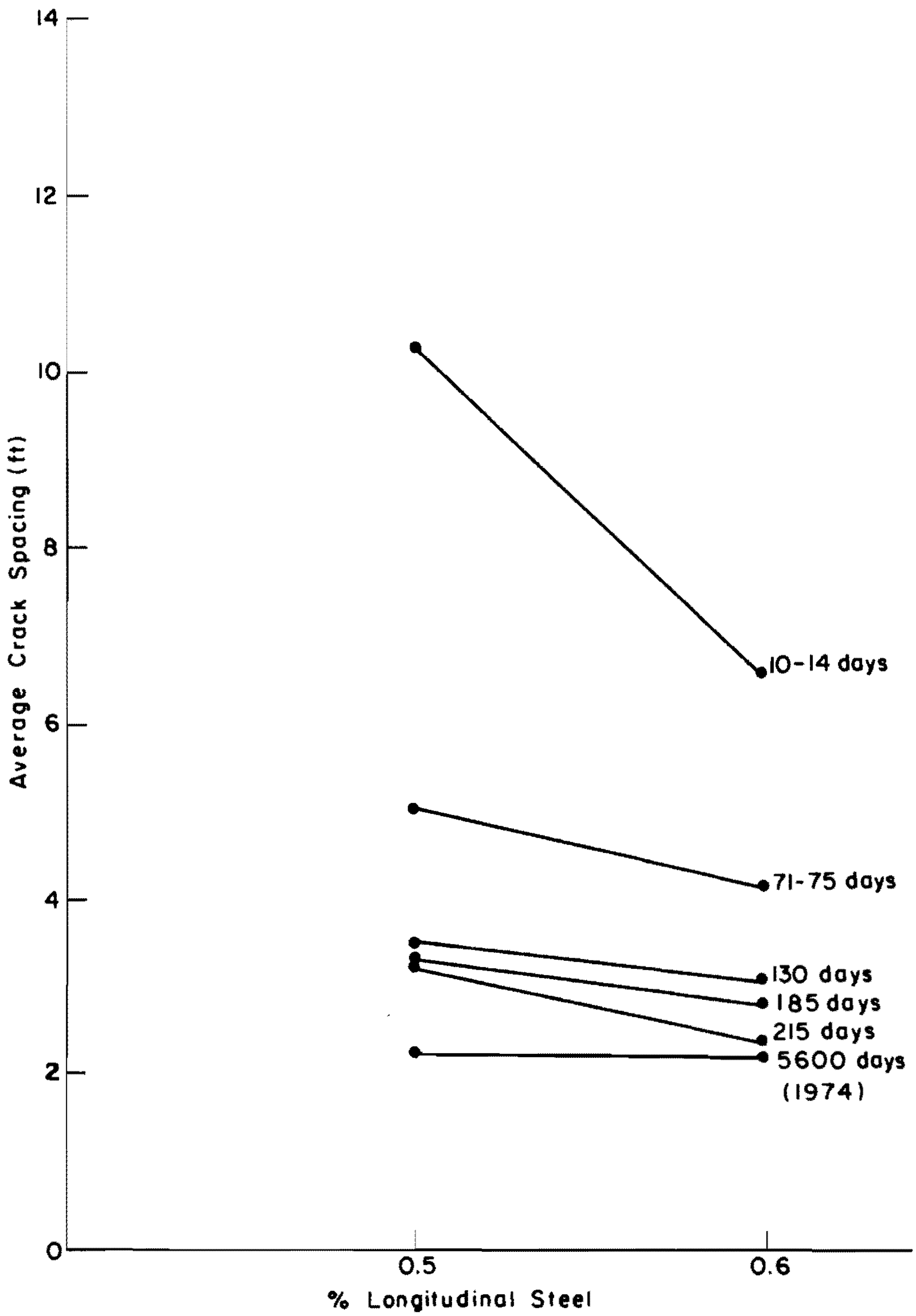


Fig 6. Plot of average crack spacing versus percent longitudinal steel.

Deflection Studies

Deflection studies have been made on this project for three different purposes; two dealt with the behavior of CRCP, and the third was concerned with the experimental overlay discussed later in this paper. The first study, in 1962, was to investigate irregularities on the project. The second study attempted to determine the effect of the percentage of longitudinal steel 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 AASHO deflection data, the pavement in these troubled areas was acting similarly to a 5.5-inch Road Test pavement, whereas satisfactory sections were deflecting similarly to a 9.5-inch Road Test pavement. The results of a subsurface investigation presented later in the paper showed that the down side of the construction joint received inadequate vibration in the lower part of the slab, which caused the bottom 3 to 4 inches of the slab to be honeycombed. As a result, the effective thickness of the slab ranged from 4 to 5 inches, and thus the data agreed with the results of the deflection study. As a result of this study, on all future jobs extra precautions were taken in vibrating the concrete on the down side of a construction joint. Additional requirements were added to the design standards and specifications.

Studies on the effect of the percentage of longitudinal steel on deflection were inconclusive. Generally there seemed to be no apparent trend that indicated sections with a higher percentage of steel had 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, limited study during the life of the facility indicated no apparent change in these observations.

Performance Studies

The roadway riding qualities were very good overall, especially when compared to the jointed concrete pavement project to the north and south of this project. However, since as early as 1962, a large number of failures have occurred in the pavement. In 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 including 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. In Appendix 2, a substantial amount of the information compiled from the field construction records is presented. Table A2.1 shows the construction data on the test sections and study sections presented in the previous sections. Table A2.2 shows a mix design and maximum temperature during concrete placements for each of the placement dates as well as the limits of placement. Table A2.3 gives pertinent information for the mix design used on the projects and can be used in connection with Table A2.2.

Construction Joint Failures. Shortly after the project opened, in 1961, serious failures were found to have quickly developed at several construction joints. Repairs on these areas revealed what the deflection studies had shown: the lower 3 to 4 inches of pavement thickness could not be counted on to act as pavement due to large and serious honeycombing of the concrete beneath the reinforcement mat. The effective depth of pavement in these areas was from 4 to 5 inches. Because hand vibrators at construction joints were not required, sufficient vibration of the bottom 3 to 4 inches did not occur in the range of 20 feet from a construction joint. In 1965, a nuclear road density logger was utilized to discover how widespread the honeycombing problem was in the pavement. Moderate success was achieved. It was predicted that 70 percent of the construction joints would fail due to the honeycombing problem, which was felt to be unrealistic at the time.

A history of construction joint failures in the NBL have been listed and included in Table 3. At the time of the overlay, approximately 75 percent of the construction joints in the northbound lane had experienced failure, which is similar to the percentage predicted 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 the prediction was reliable. Hence, the feasibility of using such equipment to identify the problem areas is reinforced. Figure 7 shows the rate of increase in failures per year and age as a linear relationship on this project. This of course would be

TABLE 3. CONSTRUCTION JOINT FAILURE HISTORY, NBL ONLY

Station	Construction Joint Condition In Reported Year					
	1961 November	1963 March	1964 July	1965 July	1969 April	1974 July
1 + 93	0	0	0	0	0	0
16 + 92	0	0	X	X	X	X
32 + 23	0	0	0	X	X	X
44 + 26	0	0	0	0	X	X
58 + 01	0	0	0	0	0	X
60 + 00	0	0	0	0	0	0
77 + 70	X	X	X	X	X	X
85 + 34	0	0	0	0	X	X
99 + 03	0	0	0	0	0	0
111 + 14	0	0	0	0	0	0
127 + 29	0	0	0	0	0	0
140 + 41	0	0	0	0	0	X
153 + 97	0	0	0	0	0	X
172 + 10	X	X	X	X	X	X
188 + 01	0	0	0	0	0	0
200 + 77	0	0	0	0	X	X
214 + 29	0	0	0	0	X	X
234 + 69	0	0	0	0	0	X
252 + 79	0	0	0	0	0	X
266 + 81	0	0	0	0	0	X
279 + 80	0	0	0	0	0	0
291 + 27	0	0	0	0	X	X
308 + 28	X	X	X	X	X	X
320 + 00	0	X	X	X	X	X
326 + 70	0	0	0	0	0	0
340 + 43	0	0	0	0	X	X
352 + 44	0	0	0	0	0	X
366 + 98	0	0	0	0	X	X
384 + 42	0	0	0	0	0	0
399 + 92	0	X	X	X	X	X
419 + 07	0	0	0	0	-overlayed-	
443 + 05	0	0	0	X	-overlayed-	
466 + 97	0	X	X	X	-overlayed-	
496 + 76	0	0	0	0	0	0
519 + 59	0	0	0	0	X	X
537 + 64	0	0	0	0	0	X
533 + 27	X	X	X	X	X	X
572 + 27	0	0	0	0	0	0
575 + 03	0	0	X	X	X	X
586 + 33	0	0	0	0	0	0
599 + 29	0	0	0	0	0	0

Note: 0 = reported as O.K.
X = reported as failure.

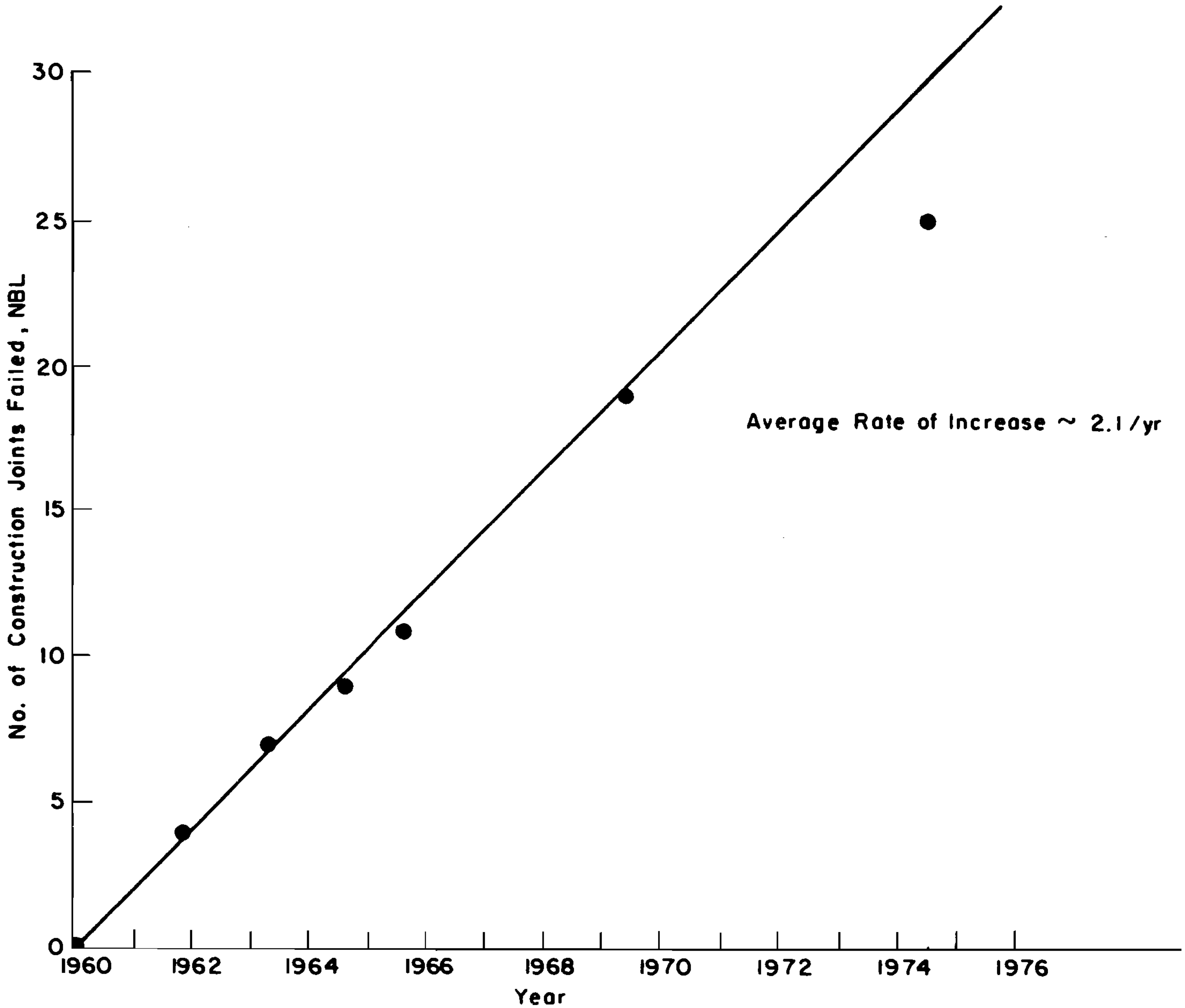


Fig 7. Plot of number of construction joints failed in NBL versus year.

correlated with the traffic buildup on the project. The average increase per year in construction joints from 1960 through 1974 has been 2.1; or, every year, 2.1 additional construction joints in this pavement fail due to excessively close crack spacing, punchouts, and spalling. Eighty-two percent (23 of 28) of the construction joints which have less than two days separation between placement experience some type of failure. Fifty-seven percent (8 of 14) of the construction joints with two days or more between pours experience failures. This difference is significant, but may be due more to general construction practices than it is to concrete, steel strength properties.

Intermediate Failures. By 1963, failures between constructions were on the rise. These failures have frequently been mentioned and studied by previous investigators. The primary "culprit" was believed to have been "flash sets" of the concrete during paving operations in hot weather. In 1964, a significant early trend was detected between percent failures in a pavement slab versus the curing temperature of that slab (Ref 2). The same type information was analyzed for the years 1969 and 1974 on the same sections. No definitive correlation exists for these years. Part of the problem in analysis is that of surveys, and will be discussed here. Otherwise, it is felt that differing weathering, soil support, traffic, and pavement strength properties over the years have had a far more significant effect than the initial curing temperature did. Naturally, a picture cannot capture all failures in a snapshot, nor can the experienced technician perfectly describe the extent and seriousness of a certain failure. The analysis of data is significantly hurt by using both methods, a visual survey one year, a picture survey the next, and no survey the third year. Although each survey can pinpoint trends in one pavement, it cannot viably be related to the magnitude of the trends in the second form of survey; thus, arises a source of "experimental error."

Table 4 presents the number of failures in terms of the longitudinal steel percentages and the roadway direction. Data from only two years are shown. It appears from the data for 1969 that substantially more failures were observed in the 0.5 percent steel than the 0.6 percent steel sections. Furthermore, there are substantially more failures in the northbound lanes than in the southbound lanes. Note that approximately 43 percent of the

TABLE 4. STEEL DESIGN AND TRAFFIC DIRECTION VERSUS NUMBER OF FAILURES FOR 1964 AND 1969.

Percent Steel	Traffic Direction	Number of Failures Reported	
		1964	1969
0.5	NB	13	186
0.5	SB	4	96
0.6	NB	4	97
0.6	SB	14	59

failures occur in the NBL and 0.5 percent combination, which is low steel percentage and high curing temperature condition, whereas, the low temperature and high steel percentage combination had only 13 percent of the failures.

Table 5 shows the percentage of the roadway experiencing failure in the northbound lanes for a range of maximum air temperatures during concrete placement. Using both the 1969 and 1974 data from the road repair survey, it is obvious that substantially more failures occurred when the concrete placement temperature was in range of 90 to 99 degrees Fahrenheit. Table 6 shows the same trend is evident in the southbound lanes although the percentages are not as high.

Experimental Overlay

An experimental asphaltic concrete overlay of varying thickness over CRCP with both percentages of steel was placed in 1969. The section overlaid was about 4650 feet long in both the northbound and southbound lanes (Fig 8). A profile of the thickness variation is given in the figure, with thicknesses of 2, 4, and 6 inches. Dynaflect readings of the sections taken both before and after overlay to determine the effects of the overlay on the load carrying capability of the pavement are presented in Table 7. Readings were taken at four different times before overlay and at three times after overlay. The data for 2-inch asphalt are plotted in terms of age in Fig 9.

A signficant 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 interrelationship (Fig 10). The readings significantly decrease with increase in thickness, as would be expected. The same data are plotted as percentage reduction of Dynaflect readings versus the thickness of overlay (Fig 11). The graph shows approximately 5 percent deflection reduction for each inch of asphalt-concrete pavement.

When the overlay was placed, the average crack spacing of the CRCP was estimated to be 2.2 feet in NBL. Since then, cracks have developed in all thicknesses of asphalt overlay. This is termed reflection cracking, i.e., cracking which is in the overlay forms at or near the crack in the CRCP.

TABLE 5. NORTHBOUND LANES - PERCENTAGE OF ROADWAY EXPERIENCING FAILURE DURING 1969 AND 1974.

Curing Temperature, °F	1969*	1974*	1974**
40-49	3.45	3.26	2.74
50-59	1.56	2.53	5.10
60-69	2.94	3.70	4.39
70-79	4.83	7.04	2.93
80-89	2.72	8.72	2.79
90-99	7.49	19.03	8.80

* From actual road repairs; for a definition of failure see Ref 22

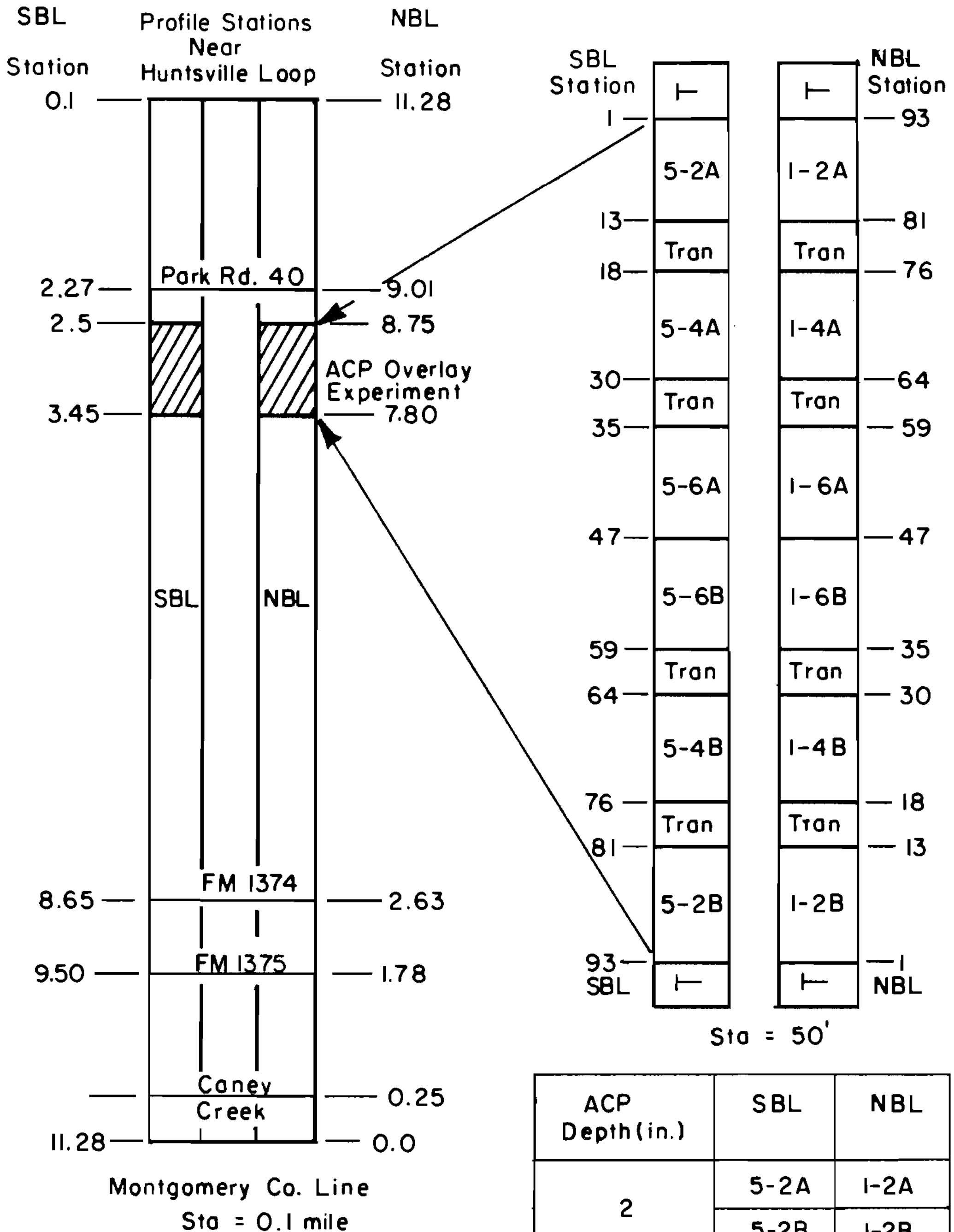
** From pictorial survey; for a definition of failure see Ref 22

TABLE 6. SOUTHBOUND LANES - PERCENTAGE OF ROADWAY EXPERIENCING FAILURE DURING 1969 AND 1974.

Curing Temperature, °F	1969*	1974**
40-49	1.31	2.81
50-59	4.39	1.40
60-69	1.33	1.20
70-79	9.30	3.28
80-89	7.15	2.64
90-99	5.40	5.92

* From actual road repairs

** From pictorial survey



ACP Depth (in.)	SBL	NBL
2	5-2A	1-2A
	5-2B	1-2B
4	5-4A	1-4A
	5-4B	1-4B
6	5-6A	1-6A
	5-6B	1-6B

Fig 8. Details of the 1969 experimental asphalt concrete overlay.

TABLE 7. DYNAFLECT DEFLECTION READINGS - BEFORE AND AFTER OVERLAY

Date	Northbound Lane					
	Average	Standard	Average	Standard	Average	Standard
	2B		4B		6B	
Before						
6/67	.964	.172	.762	.139	.826	.176
9/67	1.079	.226	.767	.144	.856	.327
11/67	.968	.171	.756	.119	.843	.230
1/68	1.107	.247	.797	.145	.915	.402
After						
2/68	.926	.369	.593	.089	.563	.148
8/68	.924	.407	.567	.093	.518	.156
1/69	1.013	.435	.585	.090	.556	.152
	2A		4A		6A	
Before						
6/67	.775	.143	.379	.137	.835	.218
9/67	.835	.133	.899	.112	.875	.215
11/67	.935	.313	1.098	.166	1.102	.376
1/68	.921	.184	1.195	.212	1.151	.372
After						
2/68	.786	.102	.833	.132	.695	.150
8/68	.640	.124	.688	.106	.581	.124

Date	Southbound Lane					
	Average	Standard	Average	Standard	Average	Standard
	2B		4B		6B	
Before						
9/67	.820	.076	.686	.099	.668	.160
11/67	1.003	.103	.862	.115	.818	.178
1/68	.763	.074	.628	.075	.737	.138
After						
2/68	.871	.115	.671	.187	.532	.121
8/68	.680	.092	.556	.161	.482	.123
1/69	.927	.158	.622	.126	.513	.112
	2A		4A		6A	
Before						
9/67	.658	.066	.876	.134	.835	.089
11/67	.835	.089	1.158	.246	.916	.226
1/68	.814	.089	1.102	.215	.797	.180
After						
2/68	.734	.068	.753	.090	.662	.142
8/68	.599	.046	.650	.056	.542	.114
1/69	.738	.056	.784	.095	.658	.157

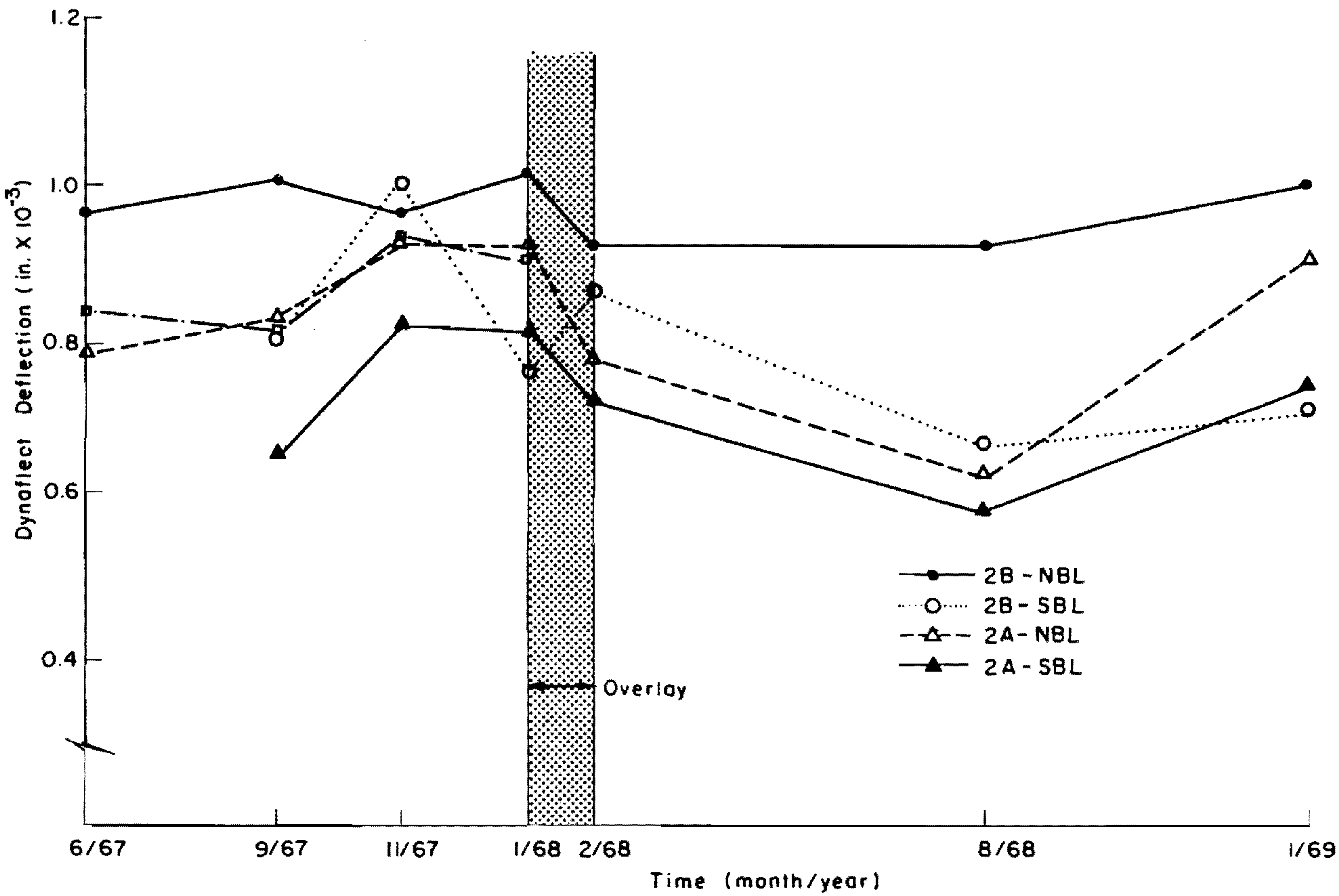


Fig 9. Dynaflect deflection readings as function of time for the 2-inch overlay.

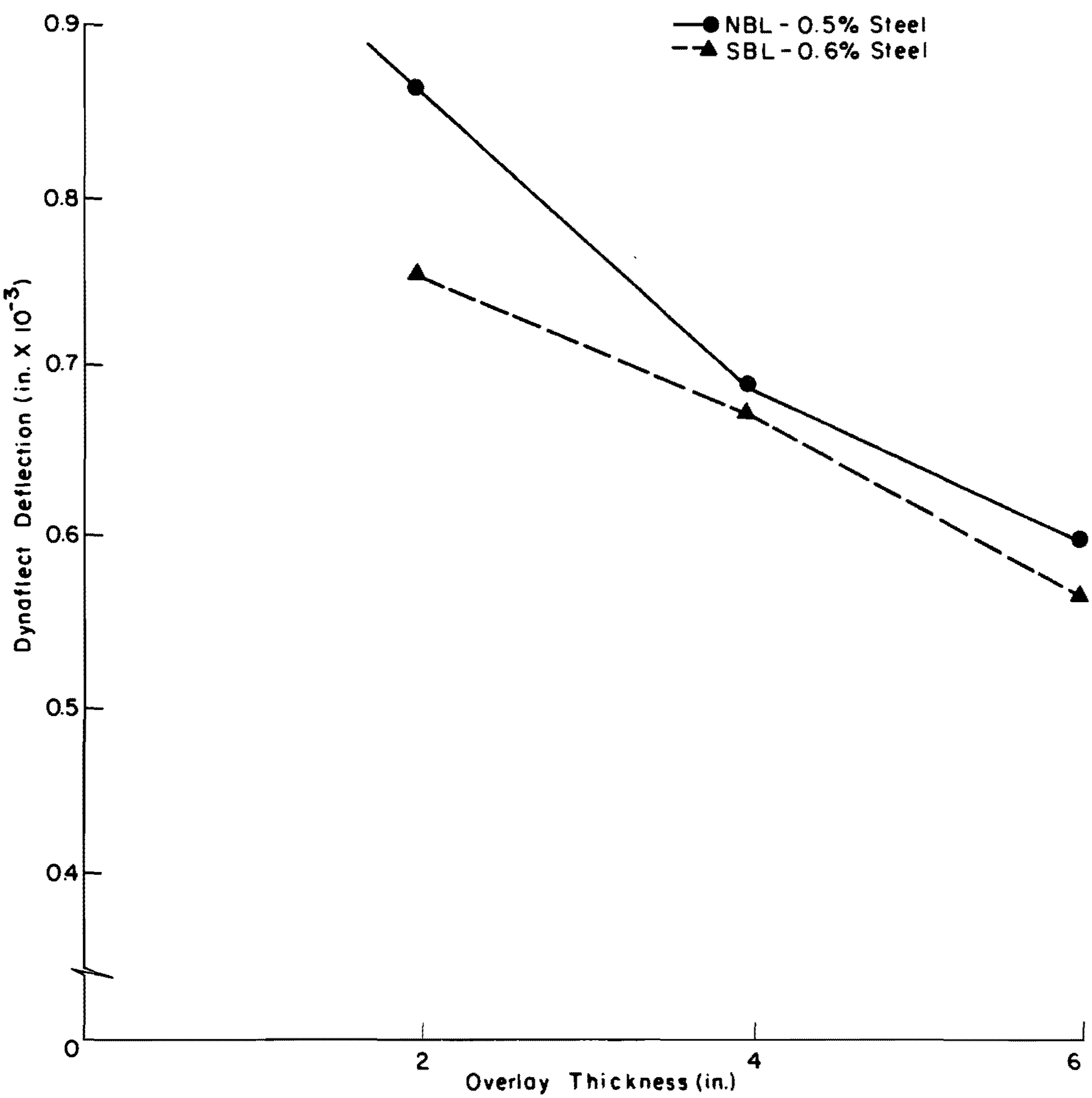


Fig 10. Plot of Dynaflect deflection readings after overlay sections versus overlay thickness.

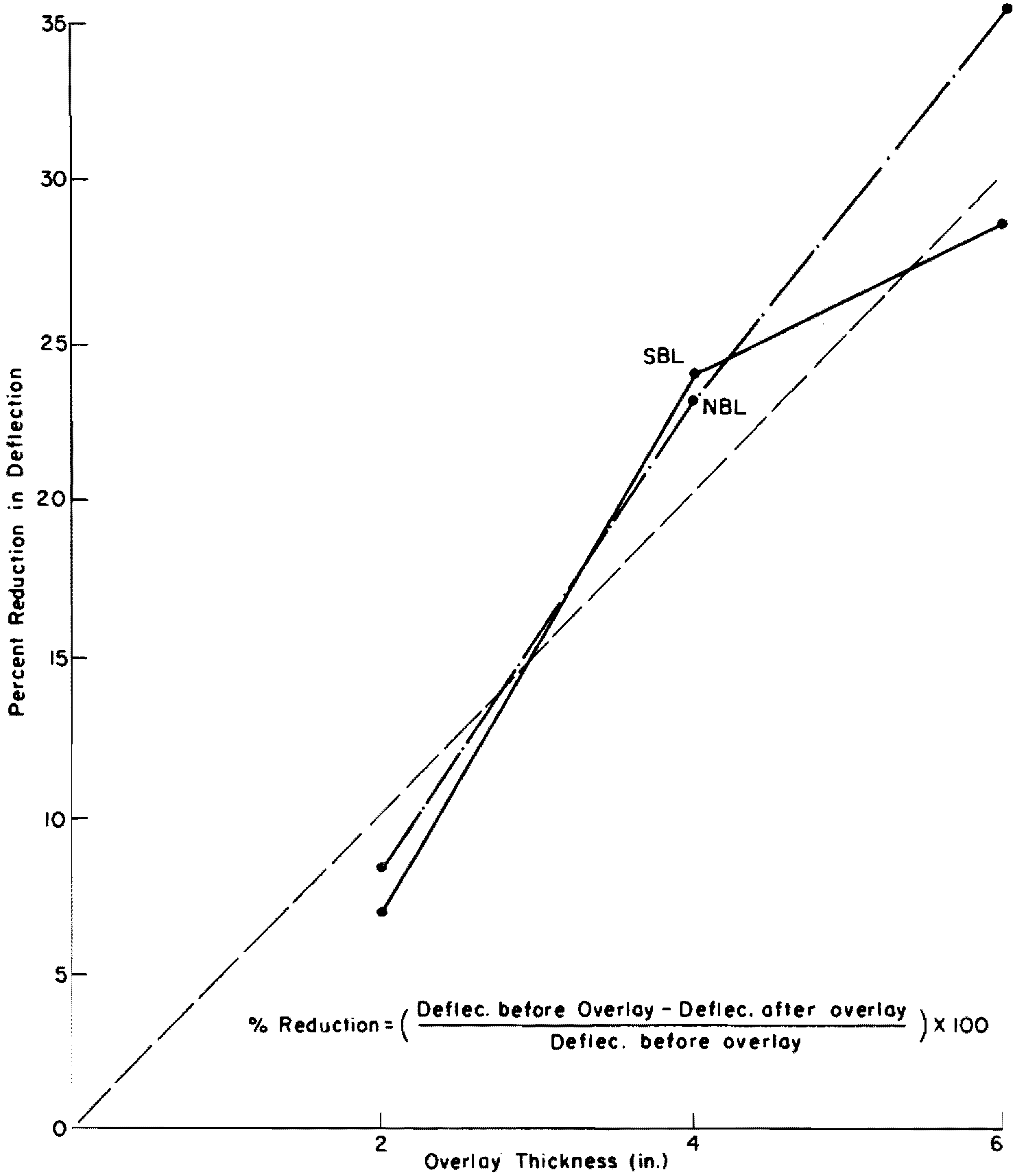


Fig 11. Plot of percentage decrease of deflection readings versus overlay thickness.

The percentage of reflection cracking is defined as the ratio of crack spacing in the CRCP in 1969 to the crack spacing in the overlay in 1974, multiplied by 100. Data for reflection cracking were available for the NBL only, as shown in Table 8. A plot of this percentage of reflection cracking versus overlay thickness (Fig 12) shows a very strong decrease in the reflection crack percentages, as would be expected. Note that for the 6-inch overlay, zero reflection cracking was experienced. A design study reported in Ref 5 indicated at least 2.5 inches of asphalt-concrete pavement was needed to prevent reflection cracking. Thus, this procedure should be modified in light of these data.

Overlay - 1974

The Walker County CRCP experiment ended in 1974 when the length of the roadway, both ways, was overlayed with asphaltic concrete. Its behavior will now be studied as a flexible pavement over a CRCP. The CRCP was put in as good a condition as possible before overlay, with all patches repaired or replaced with concrete and most failures remedied. It is hoped that this report will furnish helpful information to future studies and analysis.

TABLE 8. REFLECTION CRACKING NBL OVERLAY, 1974.

Station 416+5 to Station	Number of Reflection Cracks	Overlay Thickness, (inches)	Average Crack Spacing in Overlay-1974	Percentage of Reflection Cracking, 1969/1974
417+16	40	T	-	-
418+16	13	T	-	-
419+16	0	2	100	-
420+16	14	2	7.1	31
421+16	13	2	7.7	29
422+16	16	2	6.3	35
423+16	18	2	5.6	39
424+16	7	2	14.3	15.4
425+16	18	T	-	-
426+16	12	T	-	-
427+16	9	4	11.1	20
428+16	4	4	25.0	9
429+16	1	4	100	2
430+16	0	4	100	-
431+16	0	4	100	-
432+16	0	4	100	-
433+16	2	T	-	-
434+16	1	T	-	-
435+16	2	T	-	-
436+16	1	6	100	2
437+16	0	6	100	
438+16	2	6	50	4
439+16	0	6	100	-
440+16	0	6	100	-
441+16	0	6	100	-
442+16	0	6	100	-
443+16	2	6	50	4
444+16	0	6	100	-
445+16	0	6	100	-
446+16	0	6	100	-
447+16	0	6	100	-
448+16	0	T	-	-
449+16	0	T	-	-
450+16	0	T	-	-
451+16	0	4	100	-
452+16	2	4	50	4
453+16	2	4	50	4
454+16	1	4	100	2
455+16	2	4	50	4
456+16	1	4	100	2
457+16	8	T	12.5	18
458+16	4	T	25	9
458+78	1	2	100	2
459+78	4	2	25	9
460+78	8	2	12.5	18
461+78	8	2	12.5	18
462+78	15	2	6.7	32
463+78	9	2	11.1	20
464+78	25	T	4	T 55
465+78	16	T	6.3	T 35
466+78	25	T	4	T 55
467+78	38	T	2.6	T 84

T = transition section

Average crack spacing in CRCP at time of the overlay was 2.2 feet.

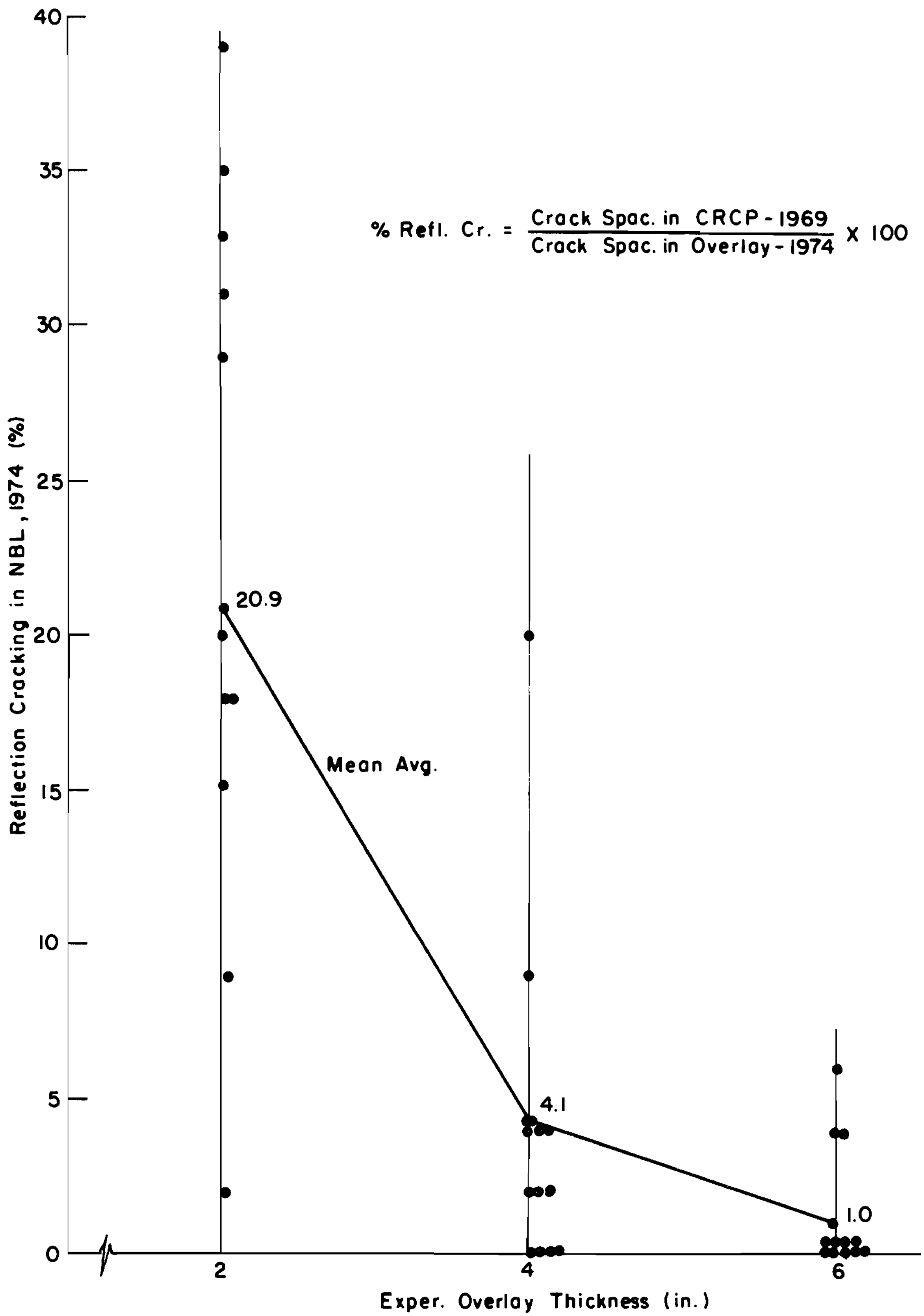


Fig 12. Plot of percentage reflection cracking versus the overlay thickness for 1974 in NBL.

DISCUSSION OF RESULTS

The sixteen-year performance history of the Walker County experimental continuously reinforced concrete pavement project provides an excellent insight into understanding the distress mechanisms associated with CRCP and also the construction and maintenance guidelines for CRC pavements. Furthermore, the importance of visible distress manifestations in the pavement to an engineer's performance rating of the pavement was emphasized, since the riding quality of the pavement always remained high even though numerous failures were present. The average present serviceability index at the time of overlay was 3.0 which is above the generally accepted minimum value of 2.5. Failures are apparent when riding through the project and have a mental effect on the rating of the performance even though the PSI is high.

In the following paragraphs, the performance of the project is discussed relative to the steps that should be taken in future design, construction, and preparation of specifications. The observations have provided valuable insight into the reasons for various levels of performance and these factors should be used in future design.

During the preparation of the plans and specifications for the project, a critical oversight was made in 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 resulted in numerous problems that showed up during the sixteen years of pavement performance prior to 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 concrete placement equipment did not adequately vibrate the concrete; hence, generally a honeycombed area was experienced in this area immediately below the steel. The thickness of the honeycombing ranged from 2 to 4 inches, and, hence, the effective depth of the concrete slab was only 4 to 6 inches. Several test procedures utilized on the project during the early life of the facility indicated that the effective depth of the slab was substantially less than 8 inches. A nuclear density device* was applied to the pavement and it was found that approximately 75 percent of the transverse construction joints had inadequate density. Subsequent pavement performance during the life of the facility required approximately 75 percent of the construction joints to be removed due to concrete density problems. In addition, deflection measurements were made on the slab during the early life of the pavement and many of the construction joints on the outside were found to perform as if the thickness was 5-1/2 inches. This checks very well with the measurements of effective slab thickness that were made during the repair operations, i.e., the thickness of adequately vibrated concrete.

Due to the experimental nature of the project, 0.5 percent and 0.6 percent longitudinal steel were used in the pavement. With the high stresses due to the fine grind cement, the effect of the percent steel showed up clearly during the performance period. There were substantially more failures in the 0.5 percent sections than the 0.6 percent sections. Recent observations on a detailed survey of continuously reinforced pavements in the state have also shown that this is true, i.e., fewer failures were evident for pavements with a higher percentage of longitudinal reinforced steel. This is not intended to imply that the reinforcement should be increased in all cases. However, the amount of reinforcement steel is a critical factor and should be carefully considered on a project basis in the future.

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 90°F or above than at lower temperatures. This was probably compounded by lack of vibration, but certainly the effect of curing temperature was evident. Possibly, consideration should be given to requiring the use of cooling water with concrete placed when temperatures are above 95°F. This is a practice followed by many state highway departments and is included in the SDHPT specifications for structural concrete. It is recommended that positive steps be taken toward control of concrete temperature during warm weather placement.

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*Lane Wells Nuclear Road Logger

Another specification requirement that eventually led to problems was the use of a maximum and minimum flexural strength for the concrete (550 psi to 675 psi) along with a minimum cement factor of four sacks per cubic yard. The contractor, in order to operate at the minimum cement factor, obtained a fine grind cement from the manufacturer. It was found later that this cement grind was equivalent to a Type 3 Cement. The result was that cracking of an explosive nature occurred during the early life of the pavement, which resulted in very high stresses of the steel and in many instances probably overstressed it. Because of this sequence of events, the specifications were revised to prohibit fine grind cements from being used with continuously reinforced concrete pavements. Subsequent performance of the pavement indicated that this was a correct step and the specification should be retained in the future.

In examining the performance of the project from an overall viewpoint, it is apparent that steel stress, average crack spacing, and the performance of the pavement were affected by the percent 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 parameters. The computer program recently developed in connection with NCHRP 1-15 accounts for many of these factors in the prediction of stresses, crack width, and crack spacing for a project (Ref 33). 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 these factors should be taken into account in design. Hence, the slabs should be designed for a range of conditions; then use a specific condition on a project basis, rather than using one pavement standard, as in the past.

CONCLUSIONS AND RECOMMENDATIONS

Based on the observations made during the performance period of the project, the following conclusions can be made:

(1) Steel stress is influenced by temperature change, crack spacing, percent steel, and cement type.

(2) 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.

(3) During the early part of the project, studies indicated that crack spacing was dependent upon percent steel, curing temperature, flexural strength, and time of placement during the day. Results at the end of sixteen years indicated that the percent 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.

(4) A study of the failures on the project indicated more failures were experienced with 0.5 percent longitudinal steel than with 0.6 percent, 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 was used and high curing temperatures were experienced.

(5) Good vibration during construction is an absolute necessity for satisfactory pavement performance.

(6) Deflection measurements before and after an asphalt-concrete overlay indicated that the deflection reduction was approximately five percent for each inch of overlay.

The following recommendations are based on the findings from the study:

(1) The maximum specific surface area requirement currently used in the specifications for continuously reinforced concrete pavement should be retained. The performance over a sixteen-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) Deflection measuring techniques and the use of a nuclear road logger may be considered on future projects to help locate problem areas, especially those with concrete honeycombing or low density.

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

(5) All the failure locations and distress areas were carefully located on the project prior to overlay. This project should be monitored in the future to ascertain the performance of the overlay over a period of time.

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APPENDIX 1

CRACK SPACING DIAGRAMS FOR 0.5 PERCENT AND 0.6 PERCENT TEST SECTIONS

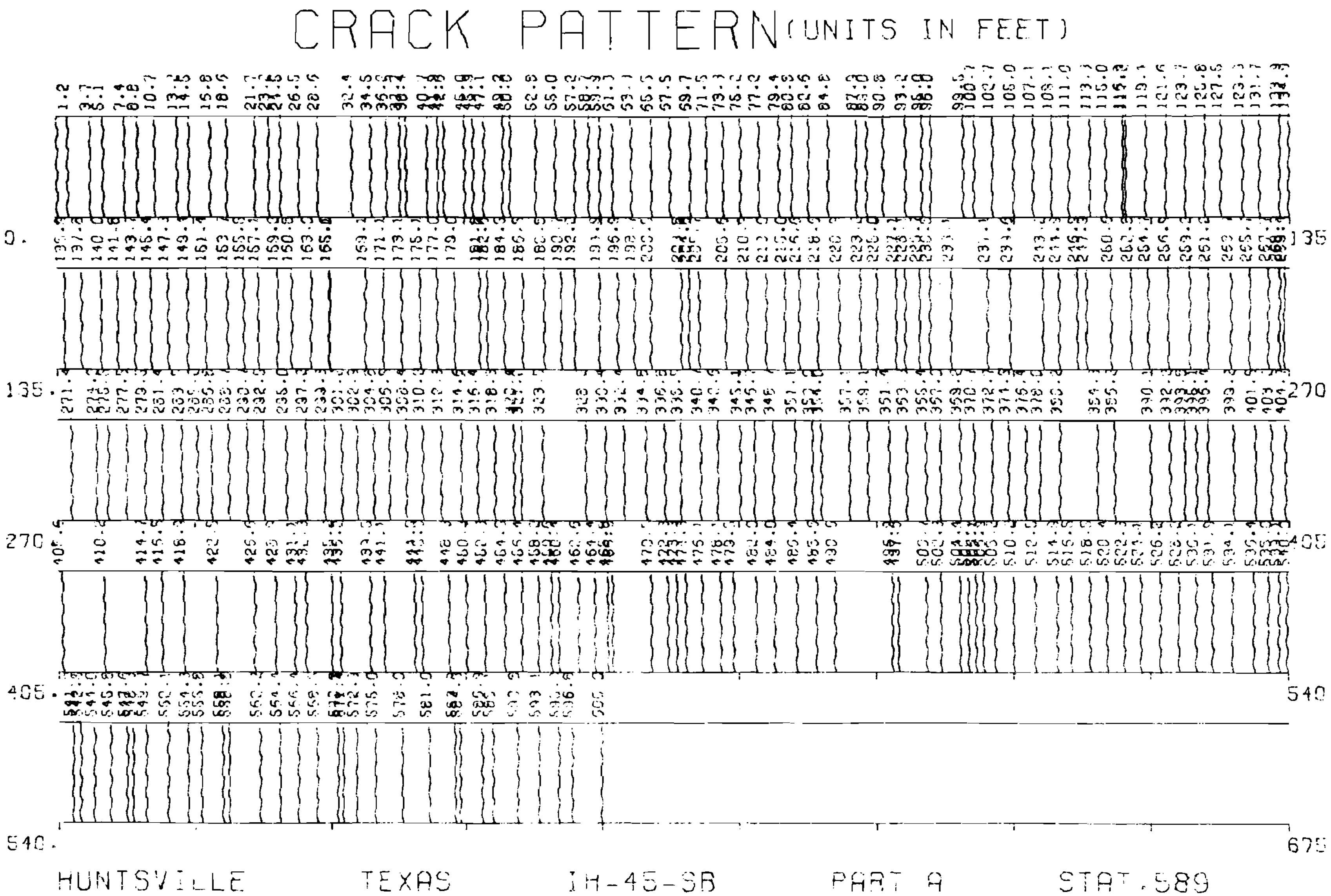

Fig A1.1. Crack pattern in SBL, from station 589+00 to station 583+00, 1974.

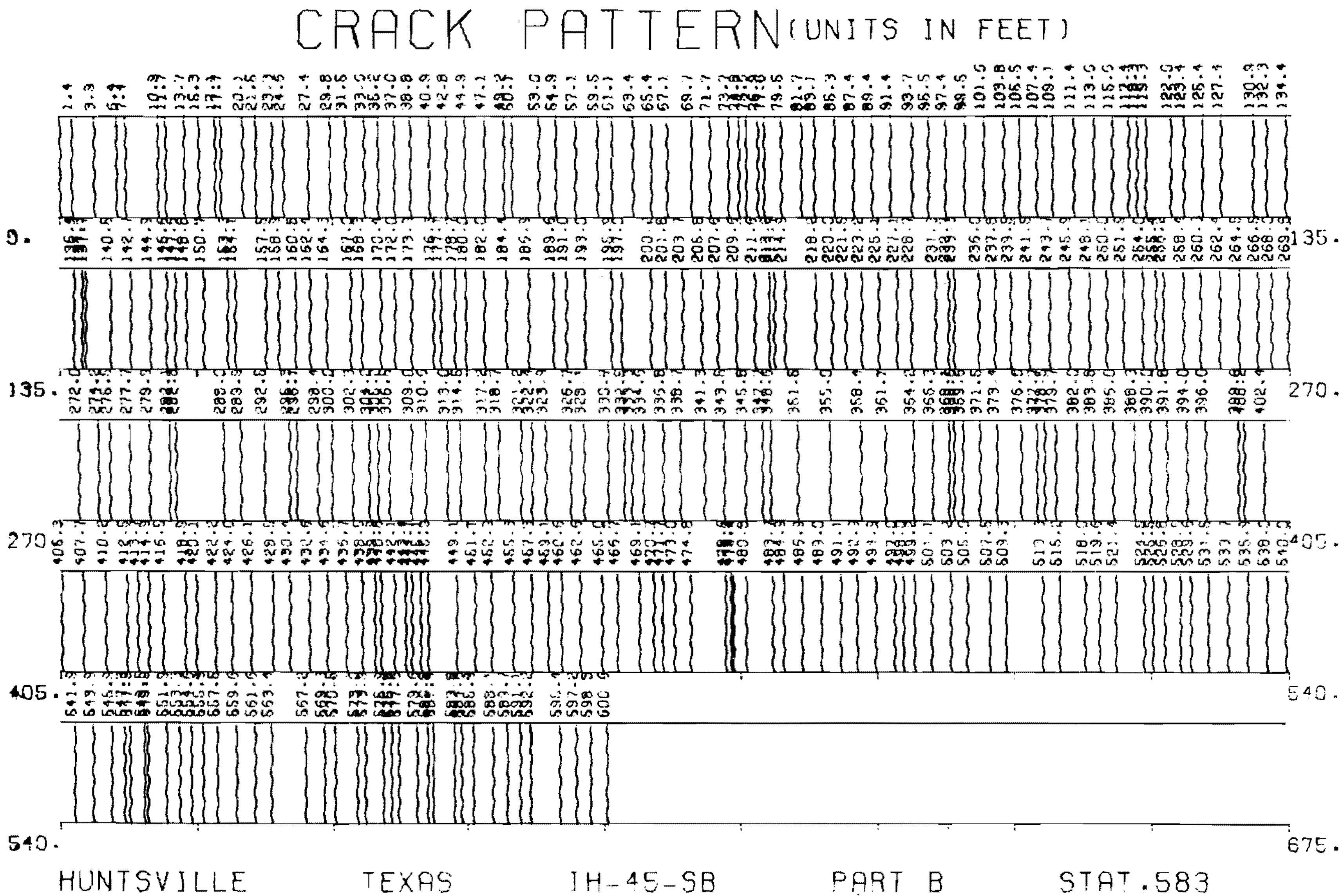

Fig A1.2. Crack pattern in SBL, from station 583+00 to station 577+00, 1974.

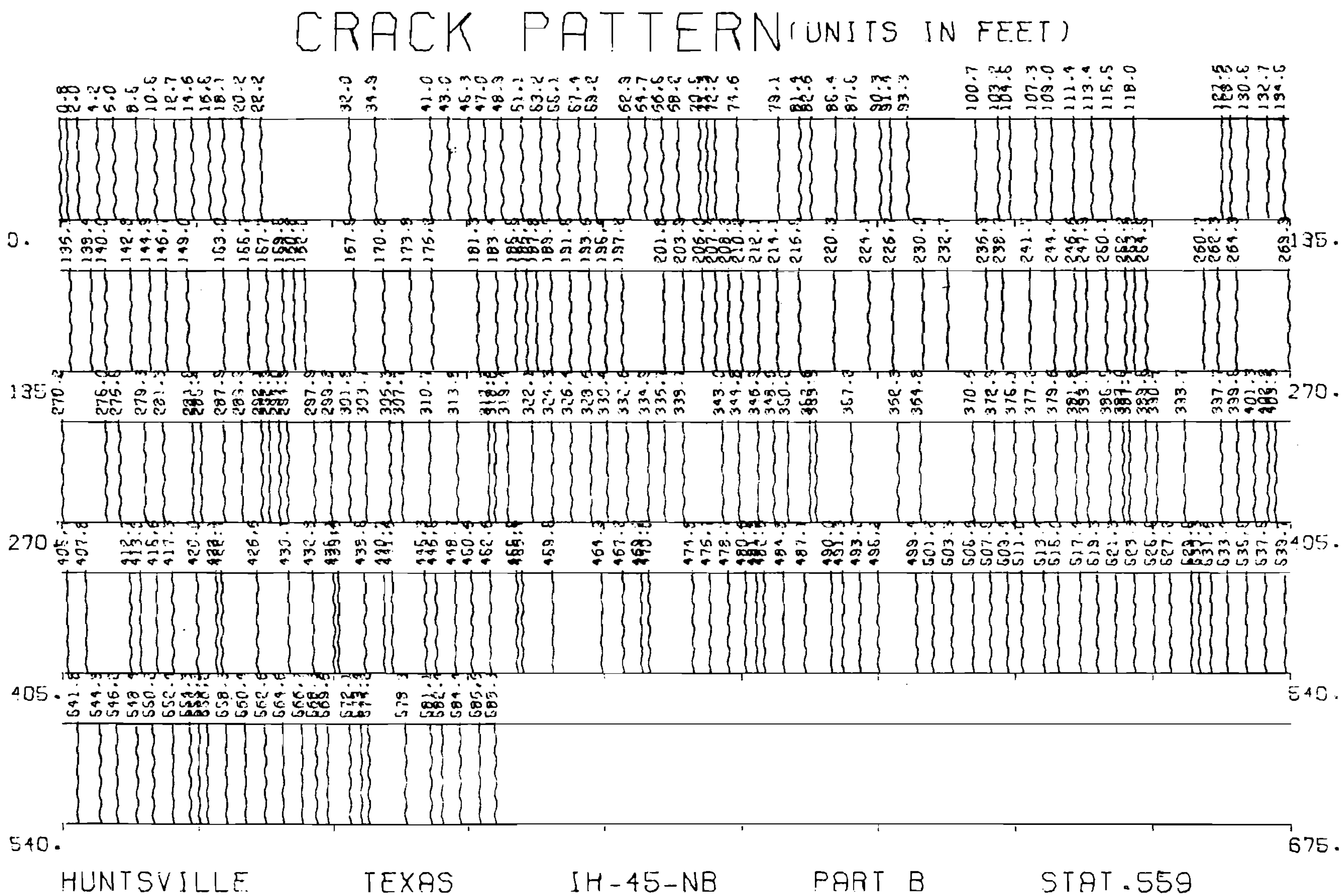

Fig A1.3. Crack pattern in NBL, from station 559+00 to station 553+00, 1974.

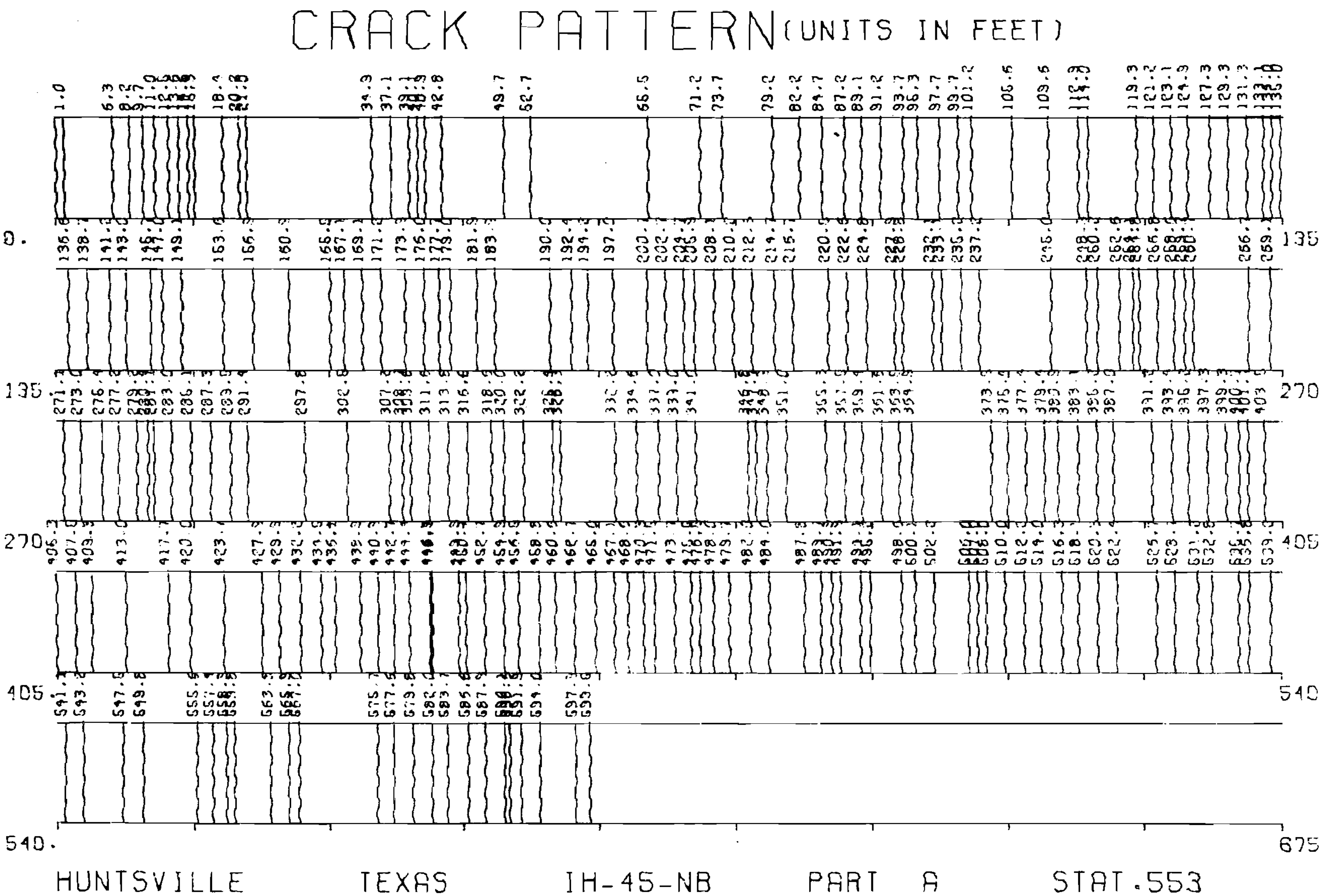

Fig A1.4. Crack pattern in NBL, from station 553+00 to station 547+00, 1974.

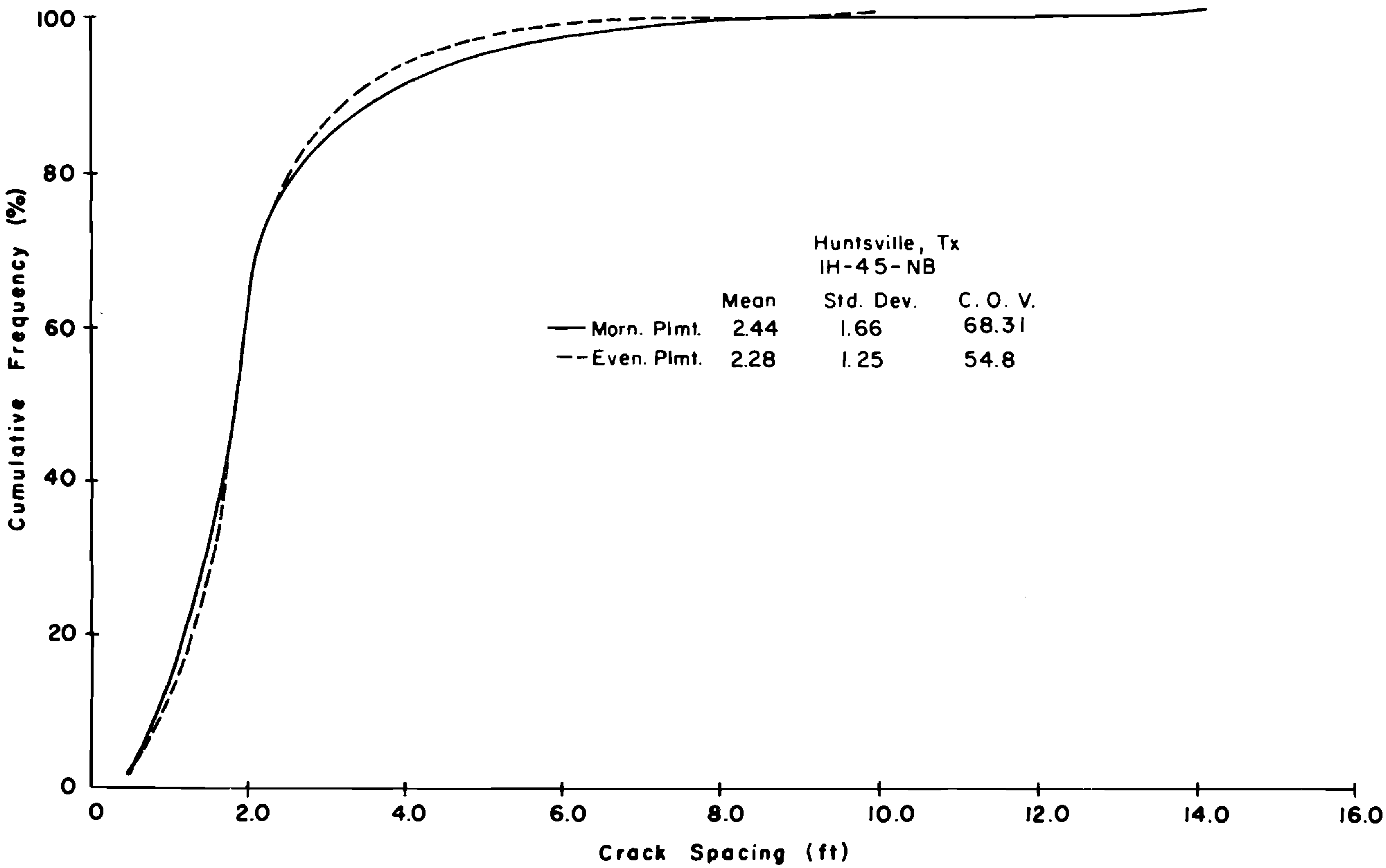

Fig A1.5. Cumulative frequency of crack spacing, NBL, from station 553+00 to sation 547+00, in 1974.

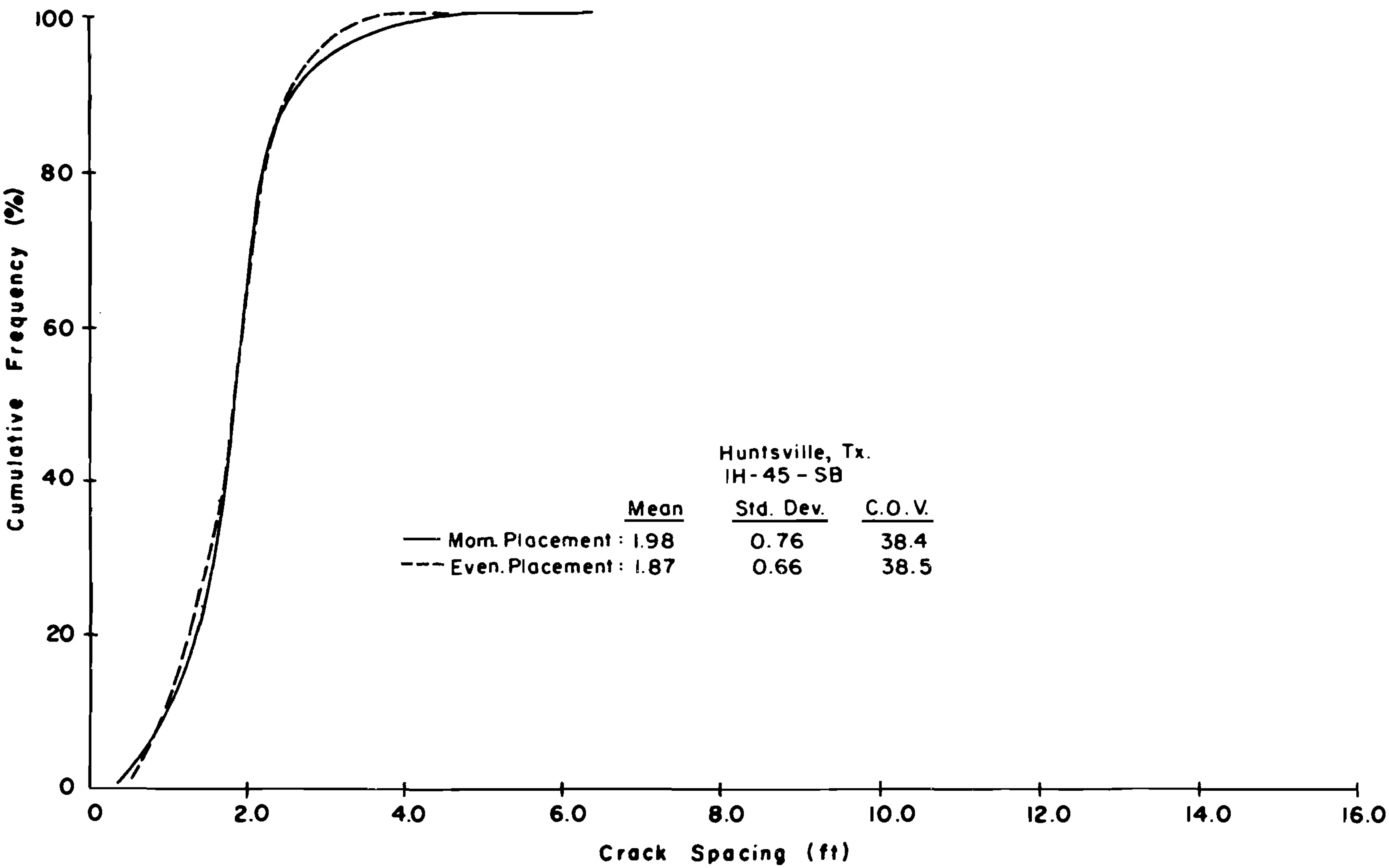

Fig A1.6. Cumulative frequency of crack spacing, SBL, 0.6 percent, 1974.

APPENDIX 2

PERTINENT DATA FROM FIELD CONSTRUCTION RECORDS

TABLE A2.1. CONSTRUCTION DATA ON STUDY AND TEST SECTIONS

	Lane	Stations	Concrete Mix Design Number	Date Poured	Highest Recorded Temperature, °F	Percent Steel	Average 7-day Flexural Strength	Cement Type
Study Section								
0	SB	565 - 569	12	8/16/60	88	0.6	556	1
1	NB	530 - 535	11	8/10/60	89	0.5	747	3
2	NB	428 - 433	10	8/3/60	93	0.5	682	3
3	SB	334 - 339	16	9/13/60	89	0.6	620	3
4	SB	298 - 303	16	10/3/60	87	0.5 & 0.6	566	3
5	SB	163 - 169	19	11/29/60	73	0.5	651	3
6	SB	99 - 105	17	12/2/60	52	0.6	749	3
7	NB	70 - 76	17	12/23/60	52	0.6	795	3
Test Section								
1	NB	533 + 27 - 572 + 23	12	8/12/60	84	0.5		3
2	SB	564 + 27 - 589 + 35	12	8/16/60	88	0.6		3

TABLE A2.2. ORIGINAL CONSTRUCTION RECORDS ON PLACEMENT OF CONCRETE SLAB

Date	Sections Poured	Mix Design	Maximum Temperature (°F)
7/25/60	320 + 00 - 326 + 70 NBL	2, 6, 1	95
26	326 + 70 - 340 + 43 NBL	1	94
27	340 + 43 - 352 + 44 NBL	1	96
28	352 + 44 - 366 + 98 NBL	1	100
29	366 + 98 - 384 + 42 NBL	1, 3	96
8/1/60	384 + 42 - 399 + 92 NBL	3, 6, 7, 8	95
2	399 + 92 - 419 + 07 NBL	9, 10, 11	91
3	419 + 07 - 443 + 07 NBL	10	93
4	443 + 05 - 466 + 97 NBL	10	93
5	466 + 97 - 481 + 34 NBL	10	95
8	483 + 73 - 496 + 76 NBL	10, 11	95
9	496 + 76 - 519 + 52 NBL	11	93
10	519 + 59 - 537 + 64 NBL	11	89
11	537 + 64 - 553 + 27 NBL	11, 12	84
12	553 + 27 - 572 + 27 NBL	12	84
15	602 + 23 - 589 + 35 SBL	12	80
16	589 + 35 - 564 + 27 SBL	12	88
17	564 + 27 - 543 + 65 SBL	12, 13	92
18	543 + 75 - 526 + 12 SBL	14, 15	92
19	526 + 12 - 511 + 02 SBL	15	90
24	511 + 02 - 494 + 84 SBL	15	94
25	494 + 84 - 483 + 00 SBL	15	94
29	400 + 69 - 472 + 27 SBL	15	86
9/1/60	472 + 27 - 452 + 56 SBL	15	91
2	452 + 56 - 446 + 39 SBL	15	95
6	446 + 39 - 428 + 45 SBL	15	91
7	428 + 45 - 408 + 59 SBL	15	92

(Continued)

TABLE A2.2. (Continued)

Date	Sections Poured	Mix Design	Maximum Temperature (°F)
9/8/60	408 + 49 - 388 + 55 SBL	15	95
9	388 + 55 - 368 + 11 SBL	15	90
12	368 + 11 - 349 + 00 SBL	15	83
13	349 + 00 - 328 + 59 SBL	15, 16	89
14	328 + 59 - 306 + 00 SBL	16	87
22	321 + 50 - 306 + 00 SBL	16	92
10/1/60	287 + 99 - 270 + 23 SBL	16	88
3	306 + 00 - 287 + 99 SBL	16	87
5	270 + 23 - 252 + 70 SBL	16	88
6	320 + 00 - 308 + 28 NBL	16	87
10	308 + 00 - 291 + 27 NBL	16	89
11	291 + 27 - 279 + 80 NBL	16	87
24	279 + 80 - 266 + 81 NBL	16, 17	85
25	266 + 81 - 252 + 79 NBL	17	80
26	252 + 79 - 234 + 69 NBL	17	79
27	234 + 69 - 214 + 29 NBL	17	79
28	214 + 29 - 200 + 77 NBL	17	82
11/1/60	200 + 77 - 188 + 01 NBL	17	68
2	188 + 01 - 172 + 10 NBL	17	74
3	172 + 10 - 153 + 97 NBL	17	79
4	153 + 97 - 140 + 41 NBL	17	83
7	140 + 41 - 127 + 29 NBL	17	65
8	127 + 29 - 111 + 14 NBL	17	67
10	111 + 14 - 99 + 03 NBL	17	73
11	252 + 80 - 233 + 03 SBL	17	76
14	233 + 03 - 215 + 35 SBL	17	78
15	215 + 35 - 196 + 29 SBL	17	80
16	196 + 29 - 179 + 66 SBL	17	80

(Continued)

TABLE A2.2. (Continued)

Date	Sections Poured	Mix Design	Maximum Temperature (°F)
11/28/60	179 + 66 - 169 + 24 SBL	17	80
29	169 + 24 - 150 + 69 SBL	17	73
30	150 + 69 - 133 + 52 SBL	17	52
12/1/60	133 + 52 - 116 + 00 SBL	17	54
2	116 + 00 - 98 + 70 SBL	17	52
13	98 + 70 - 88 + 00 SBL	17	42
19	99 + 03 - 85 + 34 NBL	17	67
22	85 + 34 - 77 + 70 NBL	17	42
23	77 + 70 - 60 + 00 NBL	17	52
27	60 + 00 - 58 + 01 NBL	17	67
1/4/61	58 + 01 - 44 + 26 NBL	17	52
17	44 + 26 - 32 + 23 NBL	18	67
18	32 + 23 - 26 + 54 NBL	18	73
18	25 + 19 - 23 + 005NBL	18	73
18	21 + 79 - 16 + 42 NBL	18	73
1/19/61	16 + 42 - 1 + 93 NBL	18	71
20	1 + 93 - 0 + 00 NBL	18	49
20	21 + 79 - 23 + 005NBL	18	49
20	25 + 19 - 26 + 54 NBL	18	49
21	88 + 00 - 74 + 97 SBL	18	62
23	74 + 97 - 57 + 31 SBL	18	52
30	57 + 31 - 44 + 82 SBL	18	45
2/2/61	44 + 82 - 32 + 65 SBL	19	70
3	32 + 65 - 21 + 46 SBL	19	68
9	21 + 46 - 6 + 49 SBL	19	50
10	6 + 49 - 0 + 00 SBL	19	65
27	602 + 23 - 599 + 29 NBL	19	73
28	599 + 29 - 586 + 33 NBL	19	66
3/1/61	586 + 33 - 575 + 03 NBL	19	61
2	575 + 03 - 572 + 27 NBL	19	66

TABLE A2.3. MIX DESIGN FROM ACTUAL CONSTRUCTION RECORDS

Mix Design Number	Water Content Ratio	Cement Factor	Percent Air	7-Day Flexural Strength	Number of Beams	Water Factor
1	6.50	4.27	3.5	731	43	7.38
2	6.09	4.02	4.0	538	2	7.09
3	6.67	4.28	3.5	679	6	7.55
4*	-	-	-	-	-	-
5*	-	-	-	-	-	-
6	6.98	4.00	3.5	774	4	7.86
7	6.89	4.00	4.0	701	3	7.89
8	7.10	4.00	4.0	748	1	8.10
9	7.19	4.00	3.0	628	5	7.94
10	7.19	4.00	3.0	657	44	7.94
11	7.10	4.00	4.0	691	35	8.10
12	6.89	4.00	4.0	664	40	7.89
13	6.85	4.00	3.0	556	3	7.60
14	6.85	4.00	4.7	670	5	8.03
15	6.50	4.00	4.7	637	99	7.68
16	6.25	4.00	4.7	618	84	7.43
17	6.25	4.00	4.7	679	222	7.43
18	6.25	4.00	4.7	676	46	7.43
19	6.75	4.00	2.5	625	29	7.38

*Not used in field.

THE AUTHORS

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