SCHOOL OF CIVIL ENGINEERING OKLAHOMA STATE UNIVERSITY

FINAL REPORT

PERFORMANCE OF EPOXY-COATED STEEL IN CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

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16. ABSTRACT

Sections of Interstate 35 in Noble and Logan Counties of Oklahoma were recently reconstructed. The reconstruction involved replacing the existing pavement with continuously reinforced concrete pavement. On both projects, northbound lanes were reinforced with epoxy-coated steel and southbound lanes were reinforced with uncoated steel. This side-by-side construction provides an excellent opportunity for comparing the performance of pavement constructed with epoxy-coated reinforcement to pavement constructed with uncoated reinforcement.

The investigation described herein included a complete crack survey on both projects, installation of half-cell sites and gauge points, construction of laboratory specimens, and comparison of measured crack spacings and widths to calculated values. Results of the crack survey indicate that average crack spacing is stabilizing between 4 and 6 ft. Average crack spacing on pavements constructed with coated steel is less than on pavements constructed with uncoated steel, but the difference is small. Initial half-cell readings indicate little or no corrosion activity on either the coated or uncoated steel. Comparison of laboratory specimens constructed with coated and uncoated steel reveals no significant difference in terms of stiffness, force required to cause cracking, or average crack width.

On the basis of the results of this investigation, it is concluded that performance early in the life of a continuously reinforced concrete pavement is not significantly affected by the use of epoxy-coated reinforcing steel. However, if the reinforcement begins to corrode, the coated steel should have a significant advantage. It is recommended that crack surveys and half-cell measurements be performed on a regular basis to assess long-term effects.

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August, 1991

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views of the Oklahoma Department of Transportation or the Federal Highway Administration. The report does not constitute a standard, specification, or regulation.

EXECUTIVE SUMMARY

Sections of Interstate 35 in Noble and Logan Counties of Oklahoma were recently reconstructed. The reconstruction involved replacing the existing pavement with continuously reinforced concrete pavement. On both projects, northbound lanes were reinforced with epoxy-coated steel and southbound lanes were reinforced with uncoated steel. This side-by-side construction provides an excellent opportunity for comparing the performance of pavement constructed with epoxy-coated reinforcement to pavement constructed with uncoated reinforcement.

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The investigation reported here was conducted as a project of Engineering Research at Oklahoma State University in the School of Civil Engineering under sponsorship of the Oklahoma Department of Transportation and the Federal Highway Administration. The authors wish to thank all who contributed to the success of this investigation, especially Mr. Mark McKinney and Mr. Michael King who worked as undergraduate research assistants. The authors also gratefully acknowledge the donation by Tulsa-based ABC Coating Company of all the reinforcing steel used in the laboratory work.

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CHAPTER 1

INTRODUCTION

1.1 Statement of Problem

Sections of Interstate 35 in Noble and Logan Counties of Oklahoma were recently reconstructed. The two reconstructed sections are both approximately 6 miles in length and involved the replacement of existing pavement with 10-in. continuously reinforced concrete pavement. In this report, the Logan County Project, IR-35-4 (115), will be referred to as Project 1; the Noble County Project, MAIR-35-4 (111), will be referred to as Project 2. Figures 1 and 2 show the locations of both projects on a state map.

Reinforcing details for these two projects, including splice lengths and steel quantities, were prepared in accordance with Oklahoma Department of Transportation (ODOT) design standards. These standards are based on the use of uncoated bars. However, the plans specified epoxy-coated bars, which the contractor supplied. Consequently, the northbound lanes of Project 1, which were the first to be constructed, were reinforced with epoxy-coated steel.

Because epoxy-coated reinforcement has been shown to have low bond capacity when compared with uncoated reinforcement, ODOT personnel were concerned that the use of epoxy-coated bars might result in unacceptable performance of the pavement. The design was therefore changed and uncoated steel was specified for the southbound lanes of Project 1. In addition, the steel percentage was increased from 0.51 for the northbound lanes to 0.61 for the southbound lanes. These design changes were also implemented on Project 2. The northbound lanes of Project 2 were constructed using epoxy-coated steel, and the southbound lanes were constructed using uncoated steel. All lanes of Project 2 are constructed with 0.61% reinforcement.

A summary of the designs for the two projects is provided in Table 1. Both projects were constructed using #6 reinforcement. In both projects, epoxy-coated reinforcement was used in the northbound lanes while uncoated reinforcement was used in the southbound lanes. Northbound lanes of Project 1 have an 8.5-in. bar spacing; southbound lanes of Project 1 and

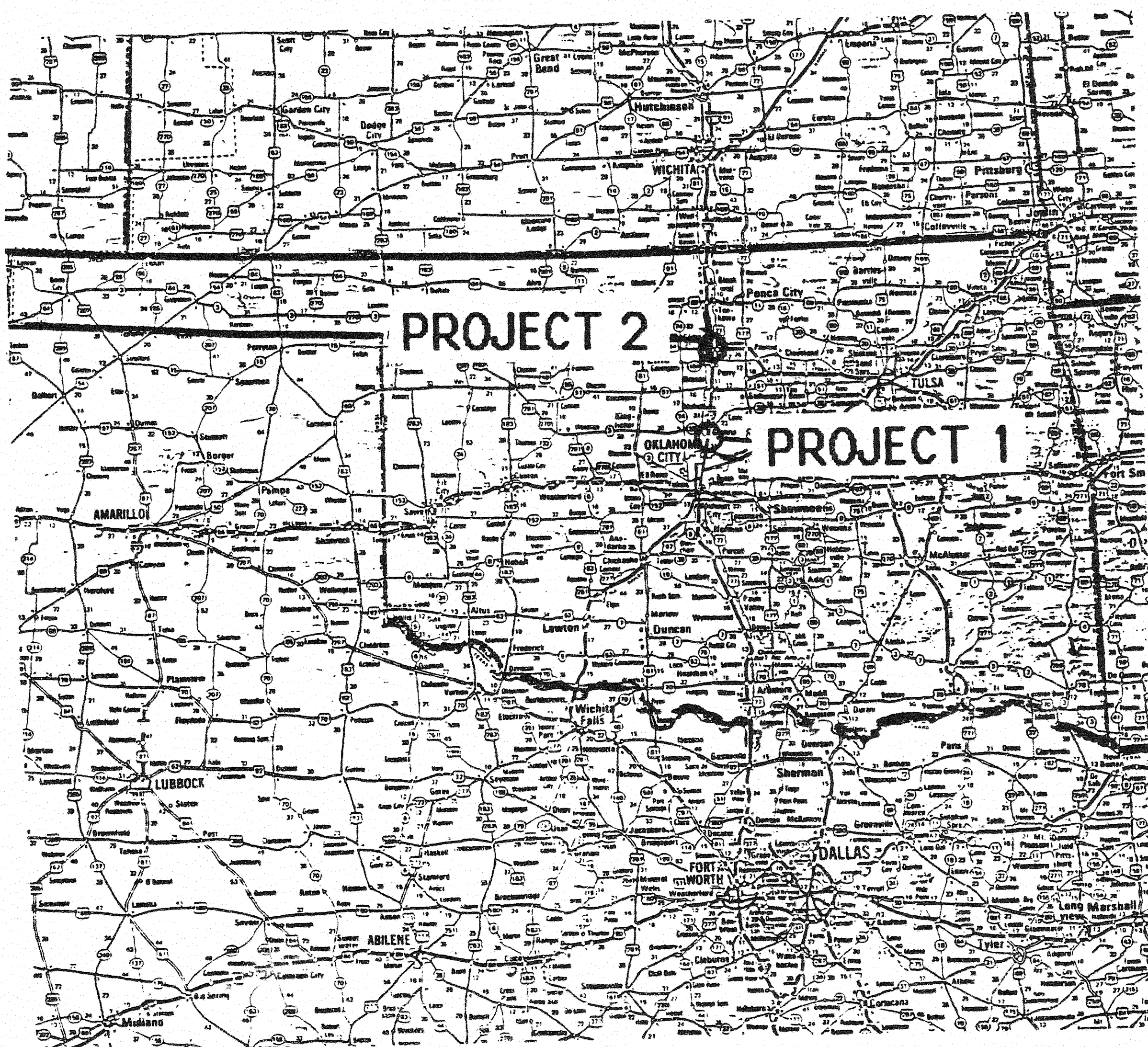


Figure 1. Locations of Projects 1 and 2 on Oklahoma State Map

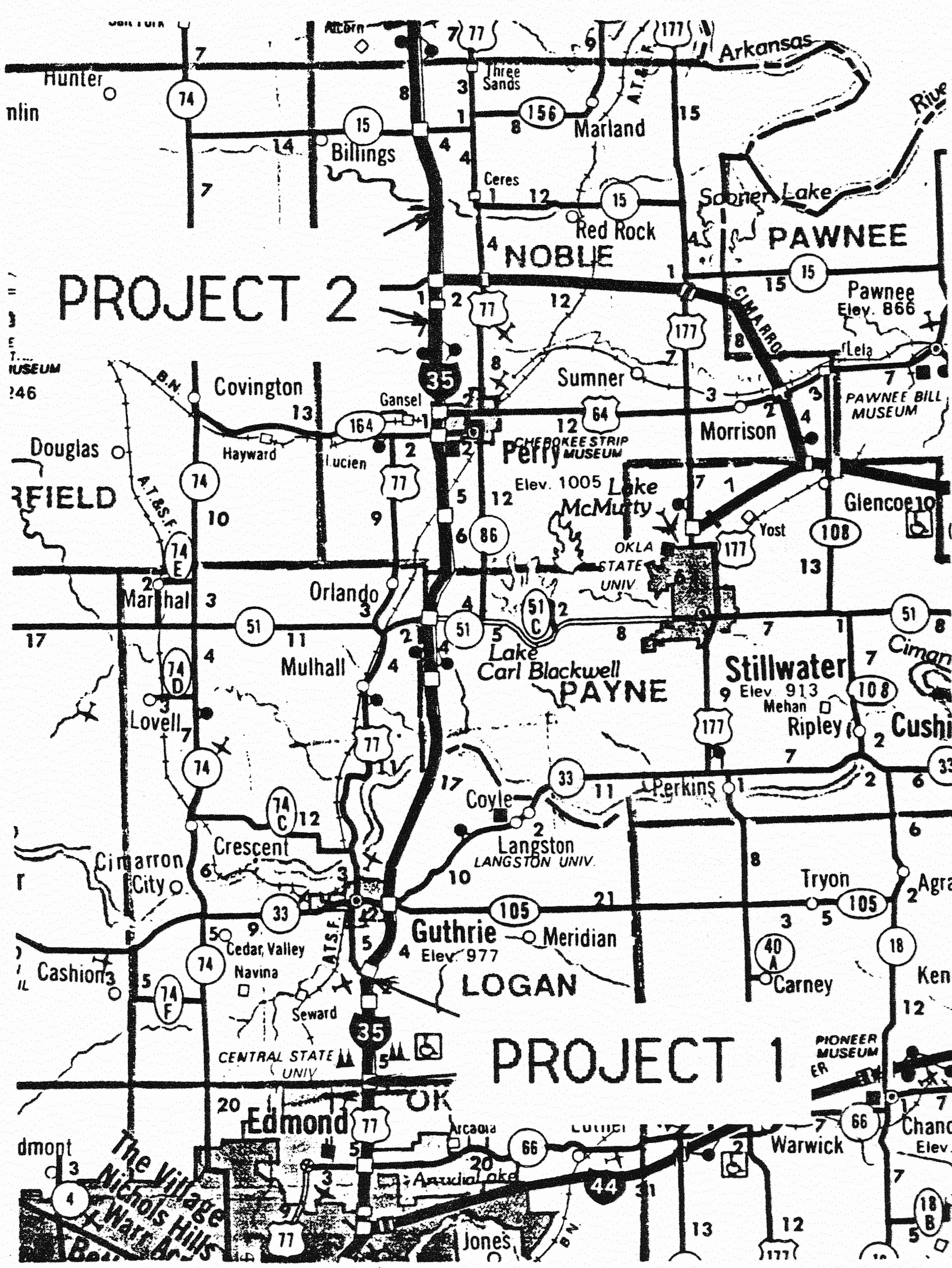


Figure 2. Map of Detailed Locations of Projects 1 and 2 on I-35

all lanes of Project 2 have a 7.25-in. bar spacing. All lanes of Project 1 and southbound lanes of Project 2 have 18-in. lap lengths; northbound lanes of Project 2 have 22-in. lap lengths.

TABLE 1. DESIGN FEATURES

Project No.	Slab Thickness	Bar Size	Spacing (in.)	No. of Bars	Percentage of Steel
Project 1					
Northbound Epoxy-coated 18-in. lap	10 in.	#6	8.50	33	0.51
Southbound Black 18-in. lap	10 in.	#6	7.25	40	0.61
Transverse bars at 44 in. c/c					
Project 2					
Northbound Epoxy-coated 22-in. lap	10 in.	#6	7.25	40	0.61
Southbound Black 18-in. lap	10 in.	#6	7.25	40	0.61
Transverse bars at 44 in. c/c					

1.2 Objective of Study

The objective of this research is to compare the performance of CRCP constructed with epoxy-coated reinforcement to CRCP constructed with uncoated reinforcement. This study includes: (1) A crack survey which involves measurement of crack spacing and crack widths; (2) installation of instrumentation (half-cell test sites and gauge points near cracks) required for future study; (3) estimation of crack spacing and cracks widths using available design procedures and comparison of these values with those obtained from crack surveys; and (4) laboratory testing of tensile specimens containing epoxy-coated and uncoated steel.

1.3 Scope of the Report

This report presents the results of a literature search pertaining to CRCP, the analysis of data from crack surveys, a comparison of values obtained from crack surveys to values estimated using available design procedures, and results of laboratory testing of tensile specimens. It also describes the installation of instrumentation required for future studies.

CHAPTER 2

SUMMARY OF LITERATURE SEARCH

2.1 Continuously Reinforced Concrete Pavements

Continuously reinforced concrete pavement (CRCP) is pavement containing longitudinal steel laid continuously without transverse joints. The only joints occur at the end of a day's work, and before and after intervening structures such as bridges. The elimination of transverse joints provides the driver with a smoother riding surface. The first continuously reinforced concrete pavement was constructed in 1921 on the Columbia pike in Virginia [11]. Since then thousands of miles of CRCPs have been laid in various states.

Since there are very few joints in the pavement, transverse cracks develop to relieve shrinkage and temperature stresses. For the pavement to operate successfully, the crack width must be small enough to prevent foreign material from entering and causing pavement growth. Crack spacing and crack width depend on the amount of longitudinal steel, concrete properties, temperature, and other environmental factors [4,5,21,25,27,28,51].

2.2 Structural Failures in CRCP

There are many factors which cause structural failures such as cracking, spalling, and pumping in reinforced pavements. Among these factors are temperature, type of subgrade and subbase, properties of concrete and percentage of steel. Construction season and early curing temperature are found to affect early crack development but not final cracking. Figures 3 through 11 [from Ref. 28] show various types of pavement failures.

In Texas, it was observed that colder parts of northern Texas experienced more localized cracking and wetter parts of eastern Texas experienced more transverse cracking. Researchers in Texas also observed considerable longitudinal cracking between transverse cracks. This longitudinal cracking, followed by spalling and pumping, led to punchouts. Researchers concluded that the major factors which influence cracking are temperature drop, shrinkage of concrete, and friction between subbase and concrete [14,25-28,52].

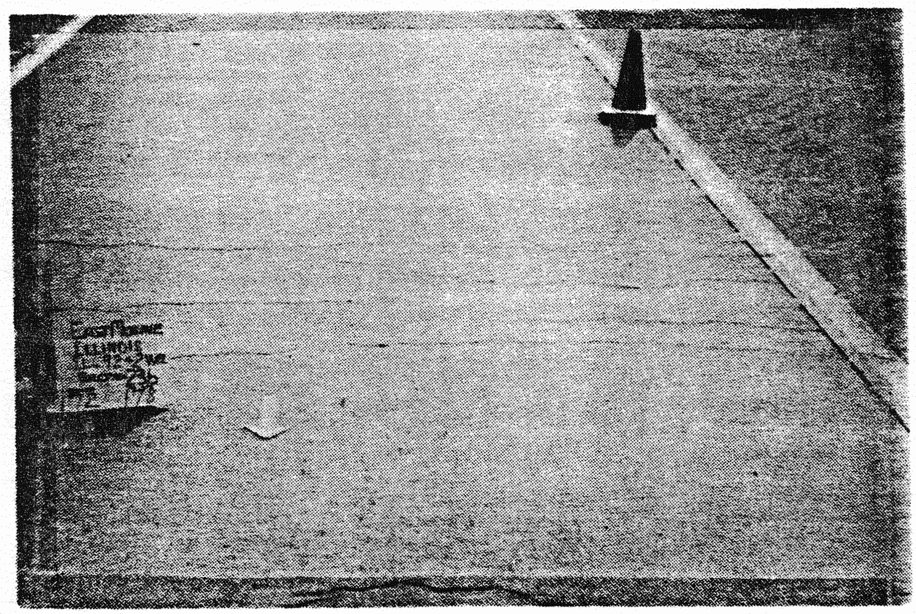
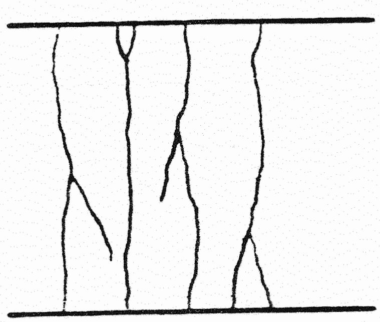


Figure 3. Minor Localized Cracking [28]



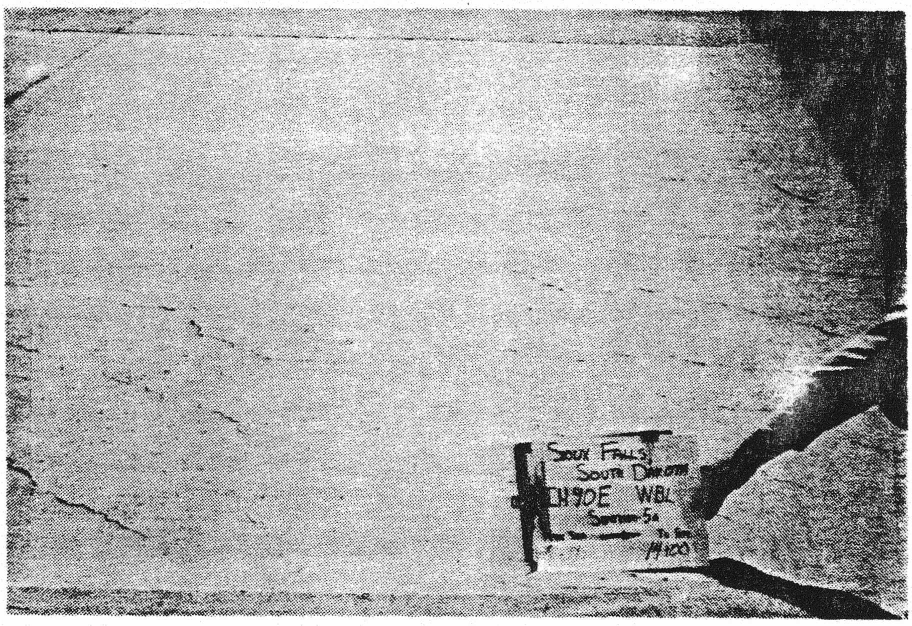
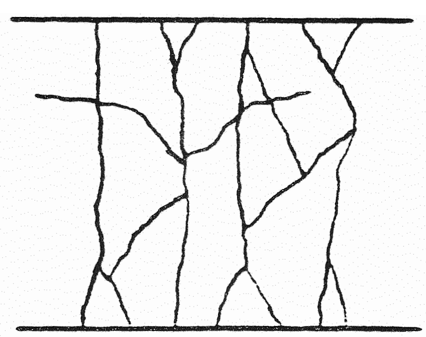


Figure 4. Severe Localized Cracking [28]



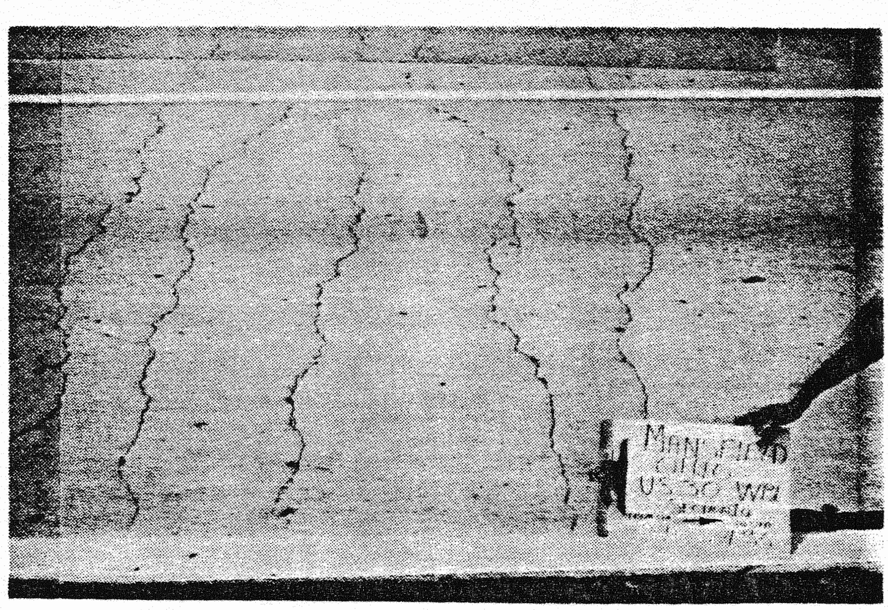
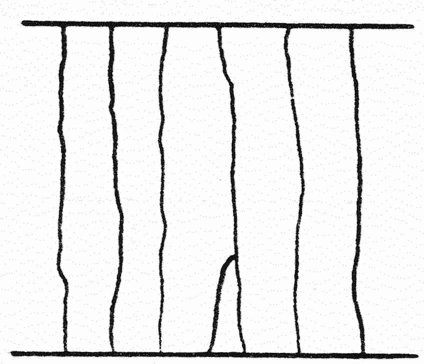


Figure 5. Severe Transverse Cracking [28]



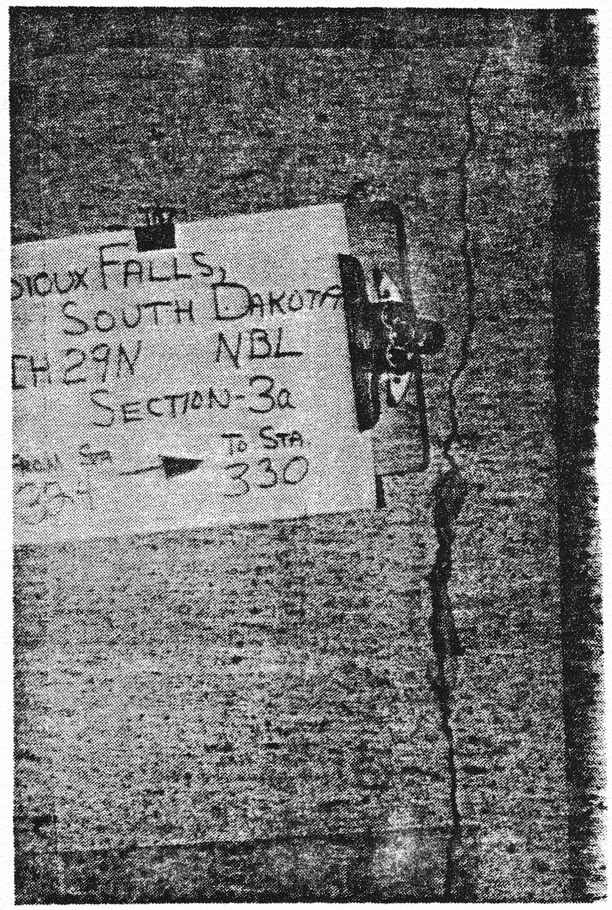
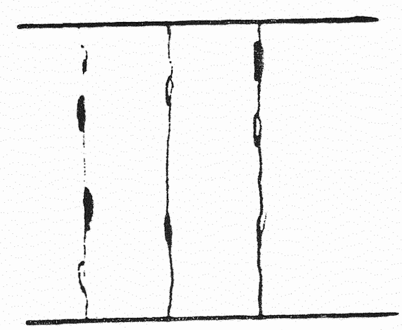


Figure 6. Minor Spalling [28]



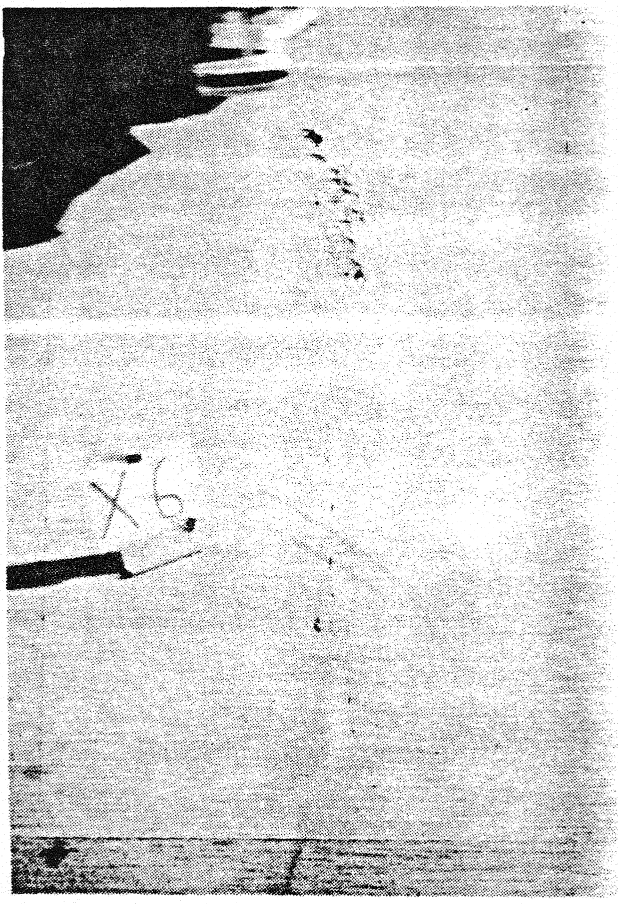
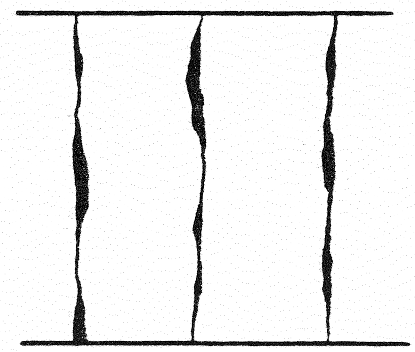


Figure 7. Severe Spalling [28]



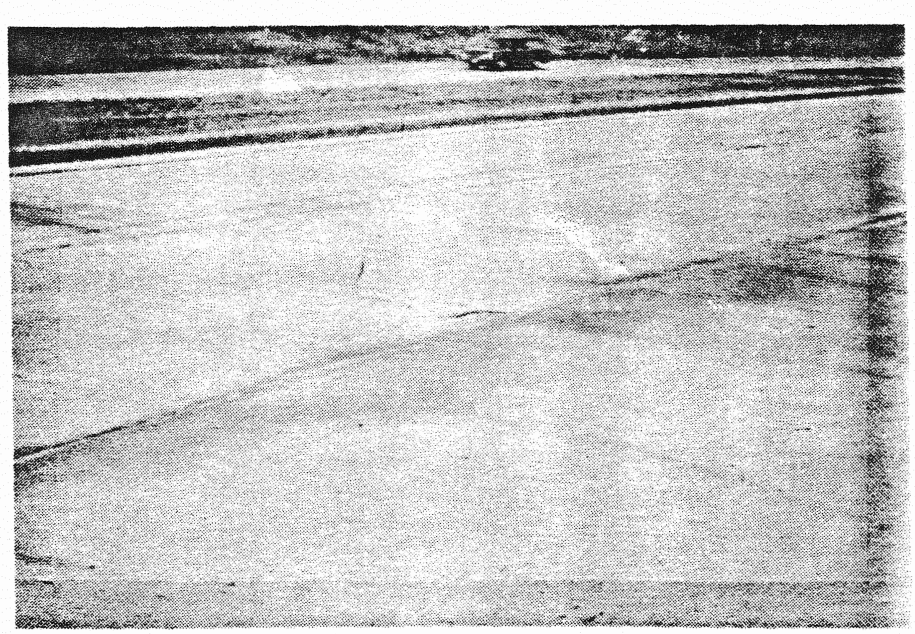


Figure 8. Minor Pumping [28]

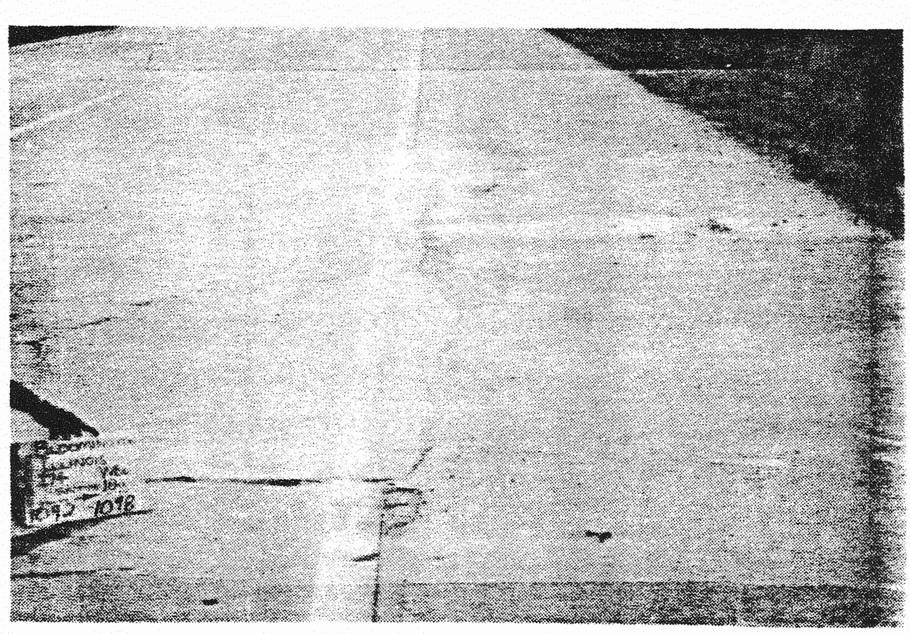


Figure 9. Severe Pumping [28]

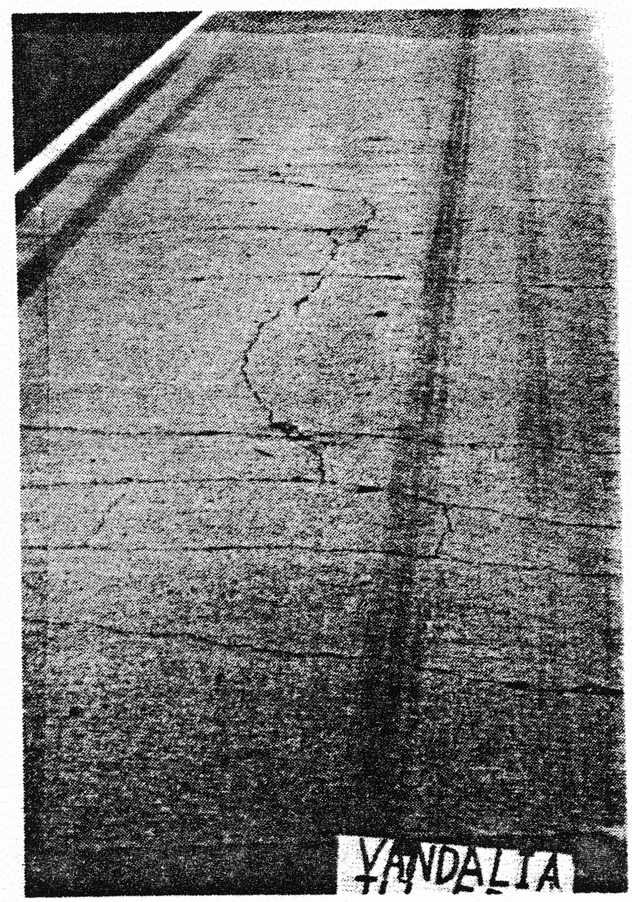
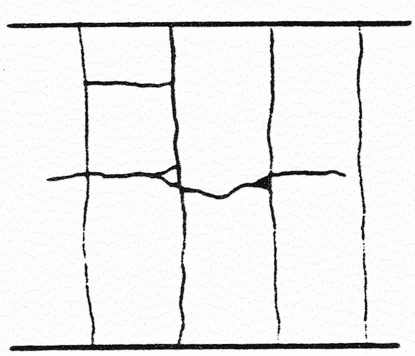


Figure 10. Minor Punchout [28]



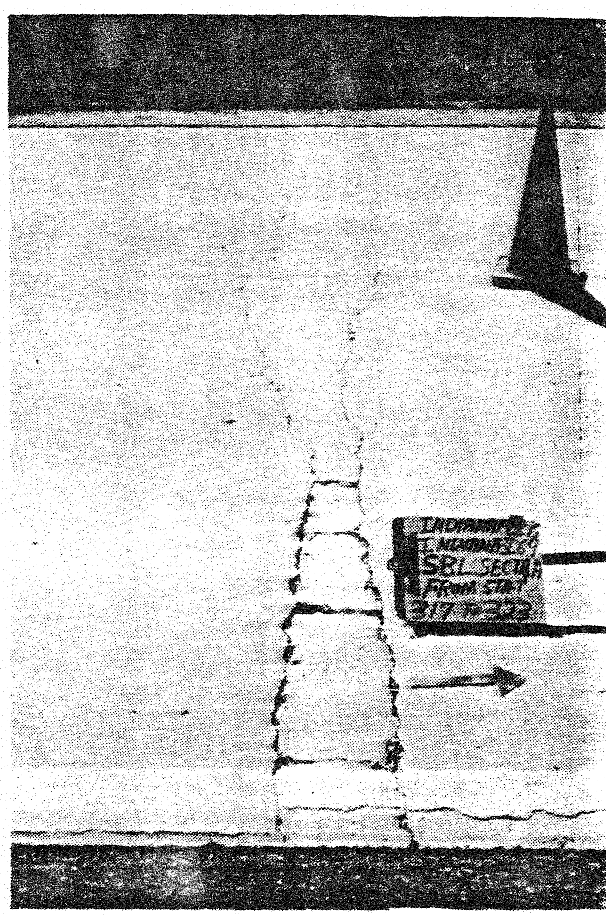
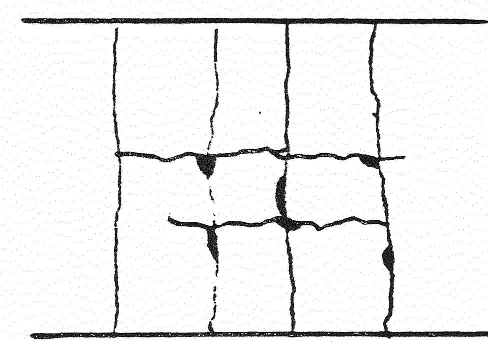


Figure 11. Severe Punchout [28]



Climate has also been blamed for poor pavement performance in Pennsylvania [51]. The affecting factors include a wide range of pavement temperatures, number of freeze-thaw cycles, frequent snow falls, and rainfalls of long duration. The effect of rainfall [5] is almost negligible for the first few years, but becomes significant once the pavement develops failures and rainwater seepage accelerates damage. More failures were observed in pavements with clay in subgrade soil due to the swelling characteristics of clay.

Percentage of steel has been found to significantly influence crack spacing and crack width in many states [4,21,27]. An increase in percentage of steel resulted in a decrease in steel stress and an increase in crack spacing. Crack width was found to be inversely proportional to percentage of steel. It was observed that cracks formed at a high rate early in the life of the pavement and then slowed as the pavement aged. The growth of cracks and crack widths was observed to stabilize after one to two years [25,52].

The absence of transverse steel is found to have little or no effect on the performance of pavement. [16,29]. In Belgium, transverse steel was skewed at 60° to longitudinal steel to reduce the risk of corrosion if cracks occurred above the transverse steel. Skewing the steel also has the effect of minimizing ripples, which can occur along transverse steel [17].

Localized failures in Illinois were found to occur predominantly in conjunction with construction deficiencies, such as absence of longitudinal steel, insufficient lap, or poor consolidation of concrete at joints [16]. Staggered lapping of reinforcing bars resulted in good joints. Terminal movement of CRCP was found to be directly related to pavement length and temperature variations.

D-Cracking, appearing as fine closely spaced cracks parallel and adjacent to longitudinal and transverse joints, has become a serious problem in Illinois [54]. D-Cracking is caused by stresses generated during the freezing and thawing of coarse aggregate and results in deterioration of concrete by the time it appears on the pavement surface. Studies indicate there is no relationship between D-cracking and corrosion of steel, but D-cracking was found to seriously affect the pavement performance.

2.3 Condition of longitudinal steel in CRCP

The most important factor affecting the performance of reinforced pavements is the condition of reinforcing steel in the pavement. Corrosion of reinforcing steel leads to deterioration of the pavement. Previous research indicates mixed experience with respect to corrosion of steel.

A 1968 study in Illinois reported a slight rusting on the surface of steel at transverse cracks [4]. Similarly, slight traces of corrosion were observed during a study in Kentucky in 1977 [21]. A more recent report from Illinois indicates an increase of corrosion activity. During a 1973 study in Illinois, cores were taken from pavements with ages ranging from 4 to 7 years [15]. Of the 151 cores taken, 74 showed no evidence of corrosion, 61 showed slight rusting, 15 showed moderate rusting, and only 1 core showed advanced rusting with a decrease in cross section area. Cores from the oldest CRCP in Illinois (placed in 1947-48) showed slight rusting in 15 or 23 cores; the remaining 8 cores had moderate rusting. Another study conducted in 1979 on the same pavement showed a considerable increase in corrosion of steel. It was also observed that corrosion is directly proportional to crack width [54].

A report from Wisconsin shows evidence of a severe corrosion problem. During a 1968 study in Wisconsin, cores were taken from two pavements to study the condition of longitudinal steel [38]. It was found that 30 and 18% of the cores on both projects showed no corrosion of steel. The remaining cores showed slight to moderate corrosion of steel. During subsequent studies conducted at two-year intervals until 1986, a continuous increase in corrosion was observed. Figure 12 shows severely corroded longitudinal steel taken from several 1984 cores. As a result of the 1984 study, it was concluded that corrosion was a serious problem in Wisconsin. Pavements with more corrosion have severe and earlier structural problems.

Since 1984, Wisconsin has used epoxy-coated steel for reinforcement. During a 1988 study, cores were taken from pavements with uncoated and epoxy-coated steel [40]: 4 cores containing uncoated steel showed visible corrosion of longitudinal steel; 2 of these had fine delamination at the level of longitudinal steel. Of the 26 cores containing epoxy-coated steel, 25 cores showed no signs of corrosion. The one core showing signs of corrosion had two bubble-like defects in the epoxy coating, indicating that with proper quality control, epoxy-

Figure 12. Severe Corrosion of Longitudinal Steel Removed From I-90-94 in Juneau County, Wisconsin

coated steel can resist corrosion effectively. Epoxy-coated steel has been found to perform satisfactorily in pavements with new as well as recycled aggregates.

2.4 Epoxy-Coated Steel in CRCP

Corrosion of reinforcing steel in concrete structures has been a major problem for many years. Corrosion is accelerated by seepage of salts into the concrete through surface cracks. These salts may be naturally present in coastal areas or they may be the result of deicing operations in the northern states. In either case, the formation of rust on the reinforcing bar produces tensile stresses in the concrete, which promotes cracking along the bar. As the crack width increases, the rusting increases, leading to deterioration of the pavement. Studies show that some pavements constructed for a service life of 20 years started deteriorating after 5 to 10 years [39,40].

Considerable research has been directed at developing methods to resist the corrosion of steel [9]. The two basic approaches are: (1) prevent the chloride in the salt from reaching the reinforcing steel by using a waterproof membrane on the concrete, and (2) protect the steel from chloride attack by covering it with a nonmetallic coating or by applying cathodic protection.

Protection of the steel by covering with an epoxy coat has been found to be effective in terms of both performance and cost. Epoxy coating is done by cleaning the bars with abrasive blasting and then coating with an electrostatic spray method. The first reported use of epoxy-coated steel was in 1973 on a Pennsylvania bridge deck [43]. Epoxy-coated steel is currently approved for use by the Federal Highway Administration and by 44 states and Canada [9]. Studies in Maryland, Minnesota, and Virginia show that epoxy-coated steel is performing satisfactorily when compared to uncoated steel [43]. In Wisconsin, the use of epoxy-coated steel in CRCP virtually eliminated corrosion [40].

The disadvantage in using epoxy-coated steel is that it results in lower steel-concrete bond strength. Tests have shown that bond strength decreases with increasing thickness of epoxy coating. Results of steel-concrete bond strength tests conducted at the University of Illinois with 7/8-in. diameter reinforcing bars suggest a minimum lap length of 30 in. or 34 bar diameters for epoxy-coated steel. This compares to a minimum lap length of 22 in. or 24 bar diameters suggested for uncoated steel [2].

CHAPTER 3

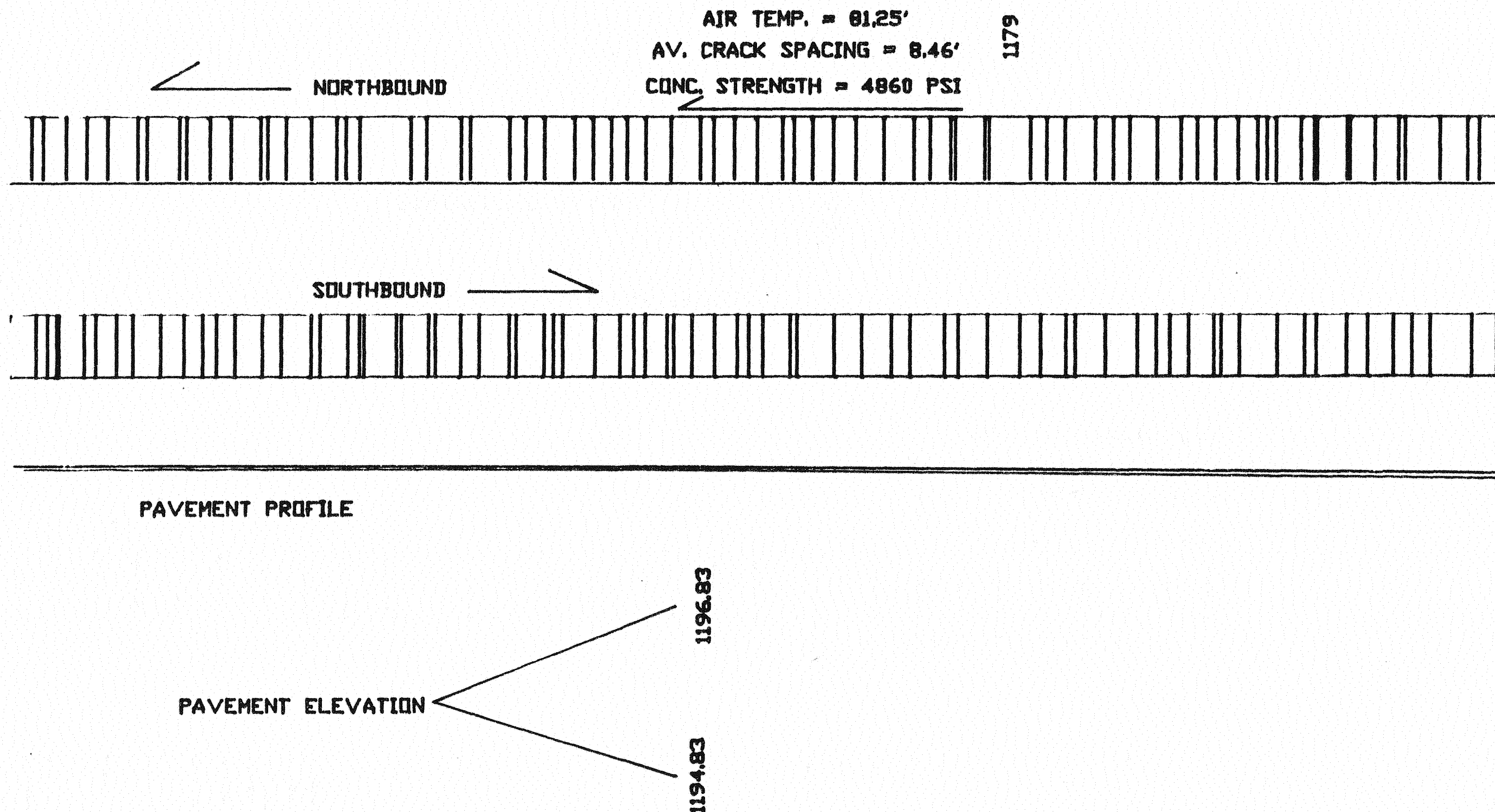
CRACK SURVEY

Two crack surveys were conducted over the full lengths of both projects. The surveys consisted of locating cracks with a measuring wheel, and recording the location and shape of each crack for later mapping. The first survey of Project 2 was conducted in May, 1989, on the northbound lanes; and in September/October, 1989, on the southbound lanes, just before opening the respective lanes for traffic. The second survey of Project 2 was conducted in August, 1990, when the northbound lanes were 15 months old and the southbound lanes were 11 months old. The first survey of Project 1 was conducted in October/November, 1989, on the southbound lanes, seven months after opening to traffic; and in November, 1989, on the northbound lanes, 13 months after opening to traffic. The second survey of Project 1 was conducted in August, 1990, when the southbound lanes were 17 months old and the northbound lanes were 22 months old.

Crack maps for typical 500-ft lengths of both projects are shown in Figure 13. The figure is based on the first crack survey. Regular spacing of cracks on Project 2 is due to the young age of the pavement. At the time of the first survey, most of the cracks in Project 2 originated at the end of transverse sawcuts made in unreinforced shoulders. In both projects, additional cracking occurred between the first and second surveys. Figure 14 shows typical cracks in the pavements.

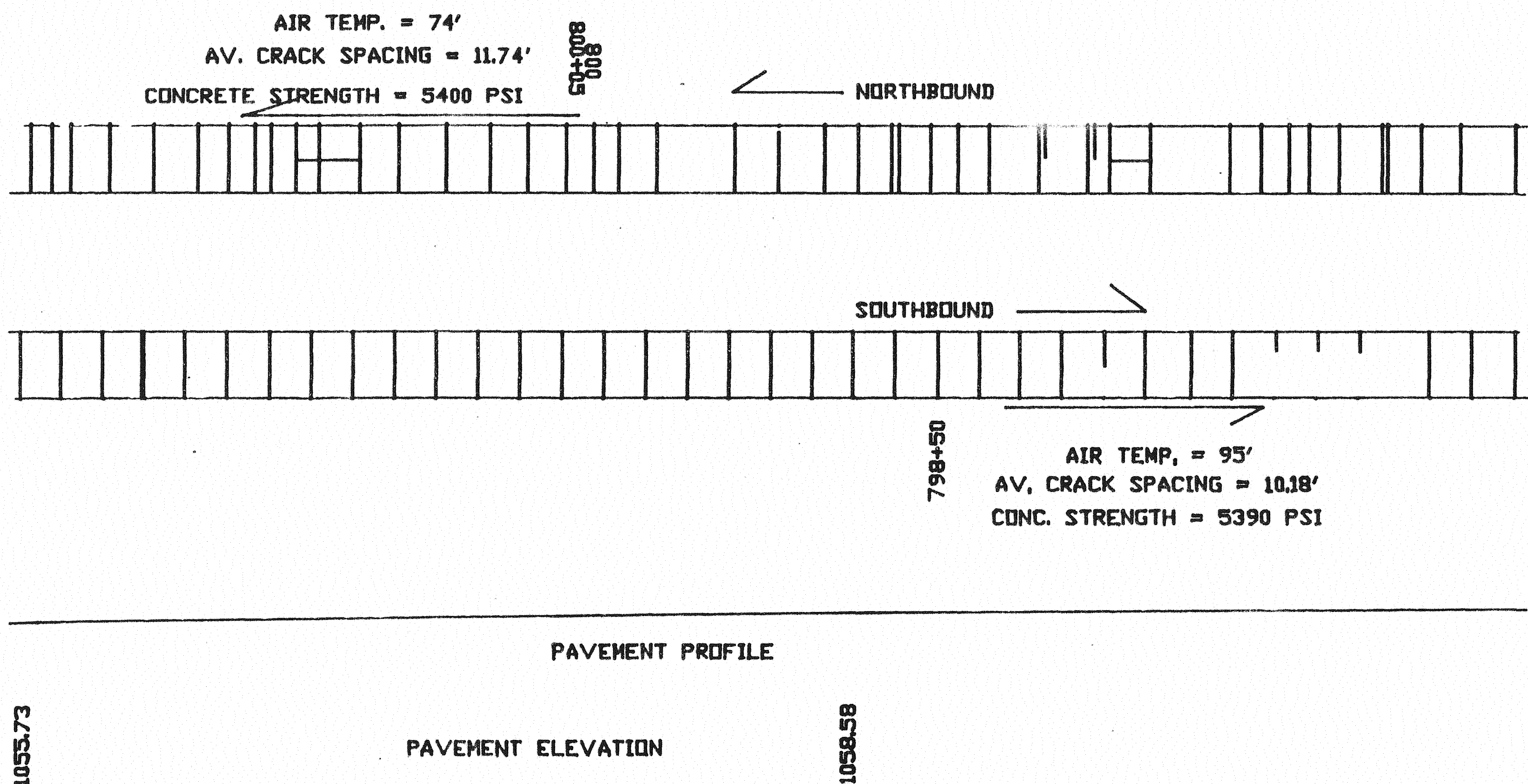
3.1 Crack Spacing

Histograms of crack spacing for surveys one and two on north- and southbound lanes are shown in Figures 15 through 22. By comparing these figures it can be seen that (1) crack spacing decreases with time (that is, more cracks develop), and (2) average crack spacing for the northbound lanes (uncoated steel) is slightly greater than average spacing for the southbound lanes (epoxy-coated steel). A statistical analysis of the data from the second survey indicates that the average crack spacings for the north- and southbound lanes are different at the 5% significance level for Project 1 but not for Project 2. The independent t-test



Project 1

Figure 13. Typical 500-Ft Sections of Projects 1 and 2



Project 2

Figure 13. Continued

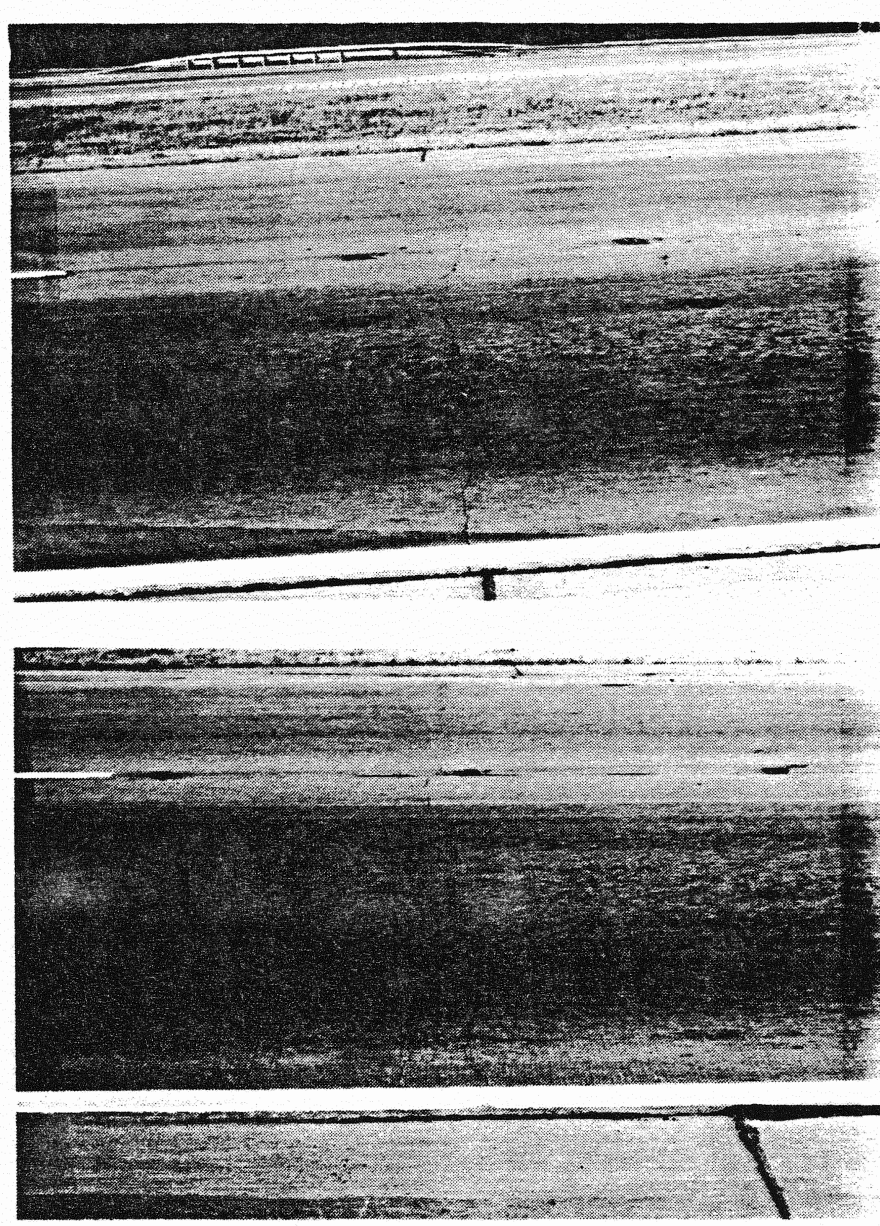


Figure 14. Typical Cracks in Pavement

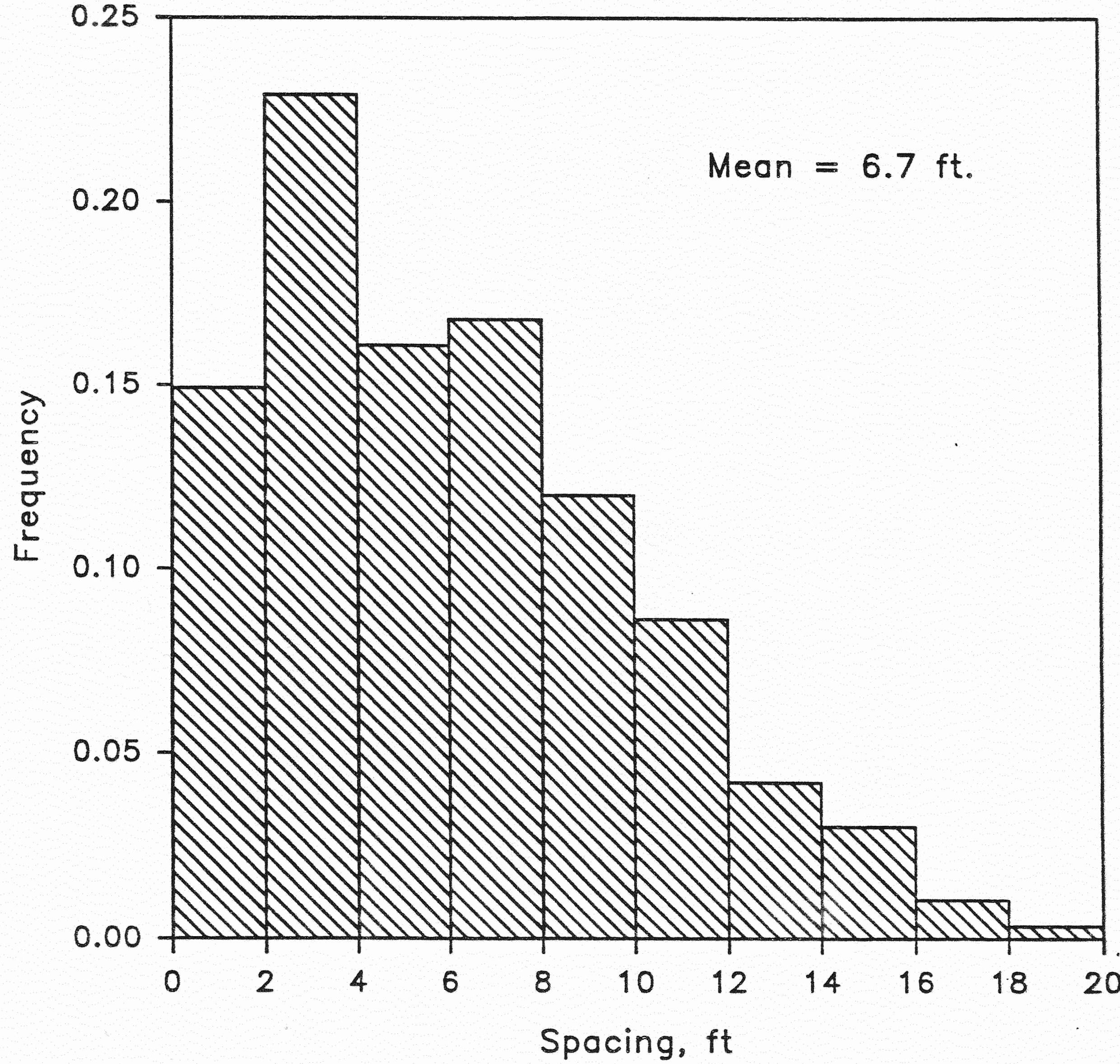


Figure 15. Histogram for Project 1, Northbound, First Survey

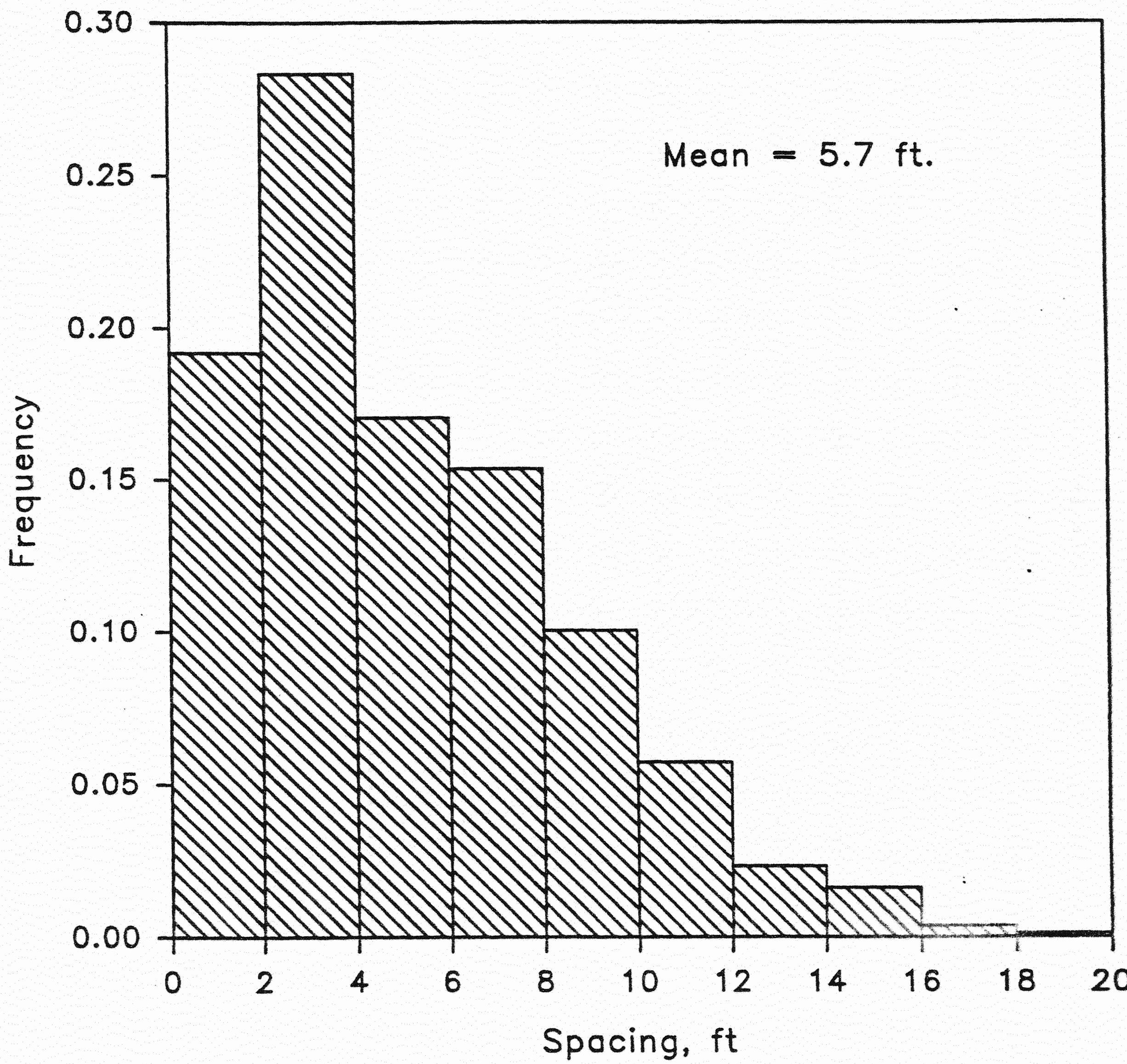


Figure 16. Histogram for Project 1, Northbound, Second Survey

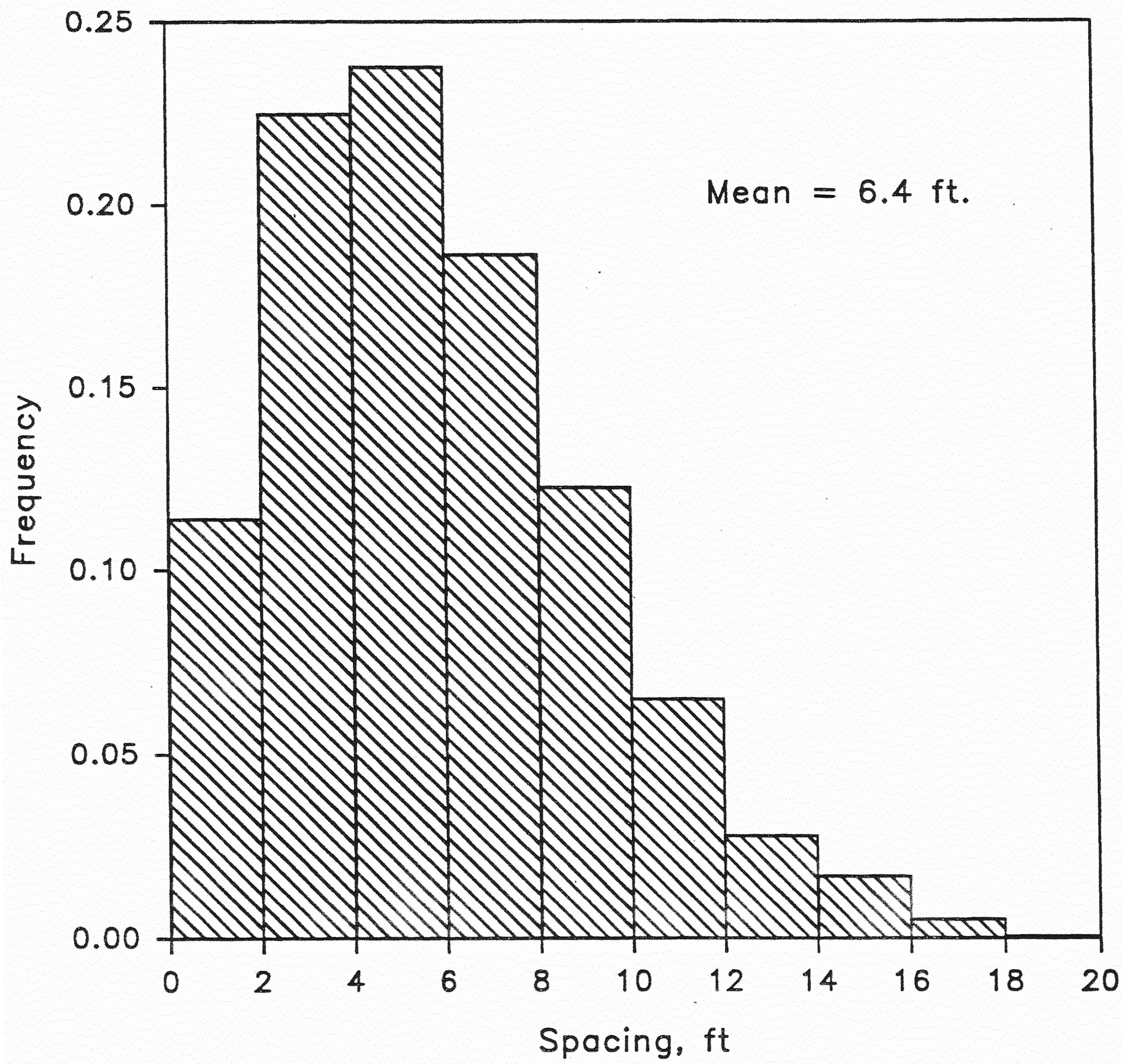


Figure 17. Histogram for Project 1, Southbound, First Survey

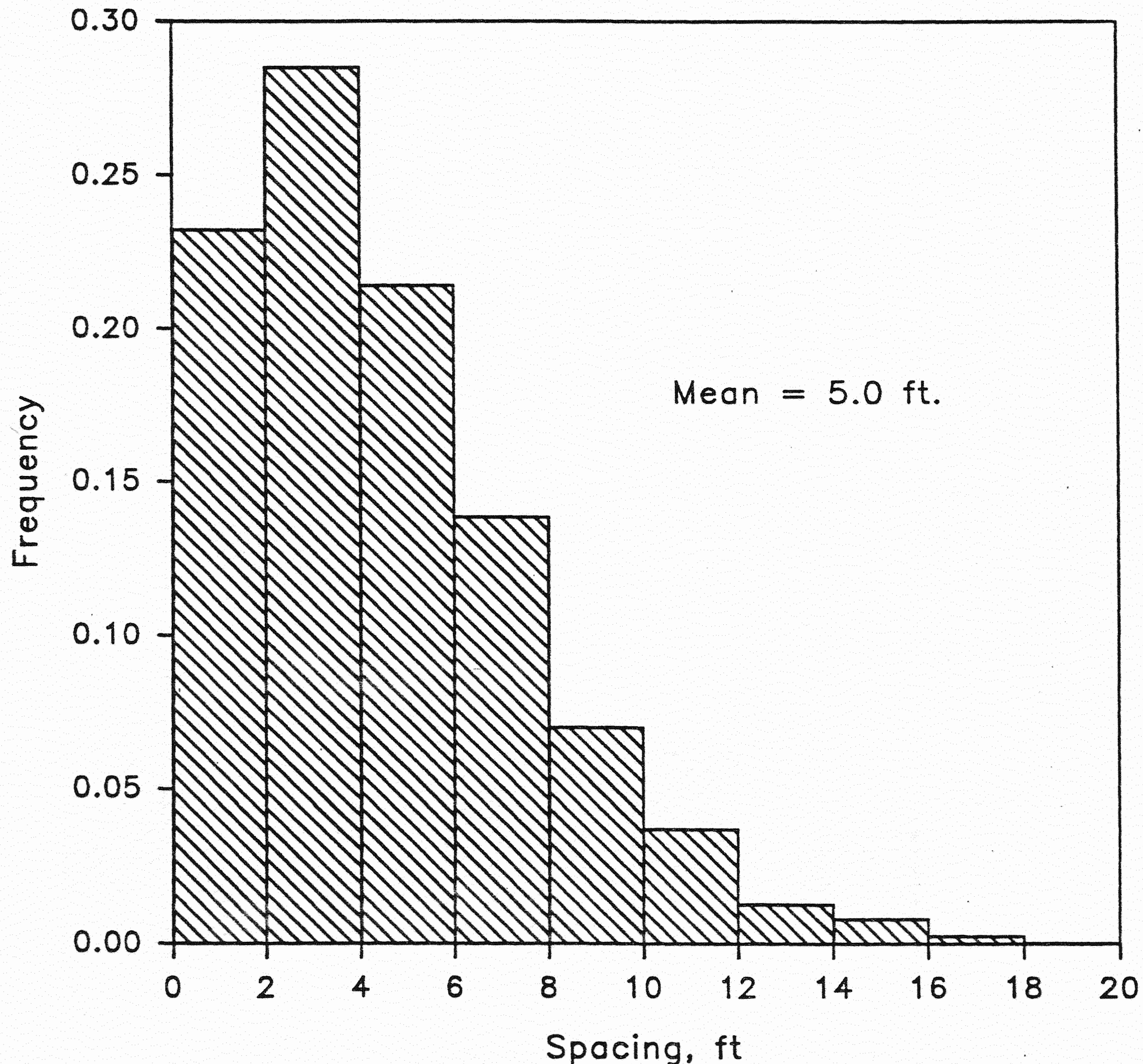


Figure 18. Histogram for Project 1, Southbound, Second Survey

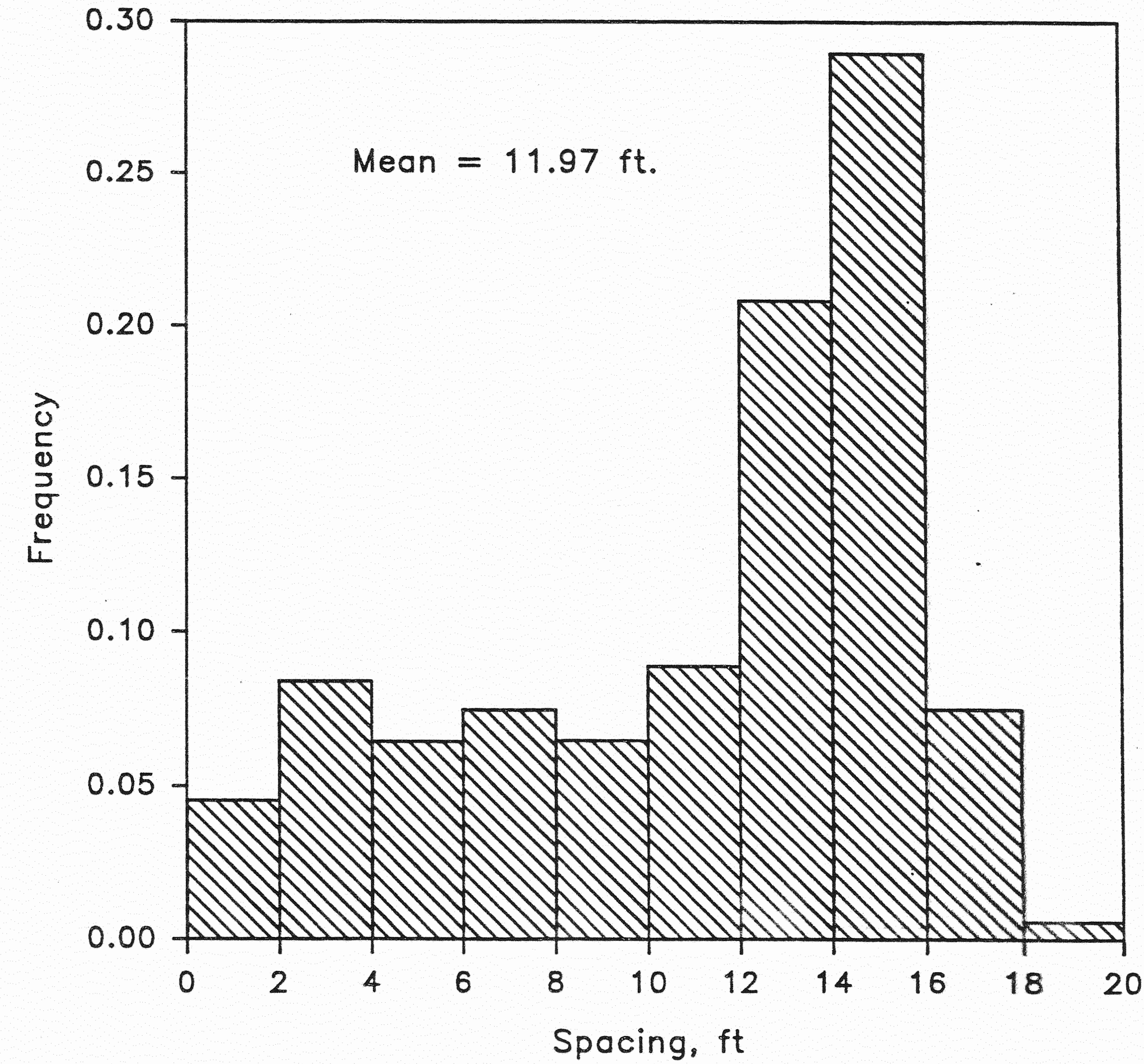


Figure 19. Histogram for Project 2, Northbound, First Survey

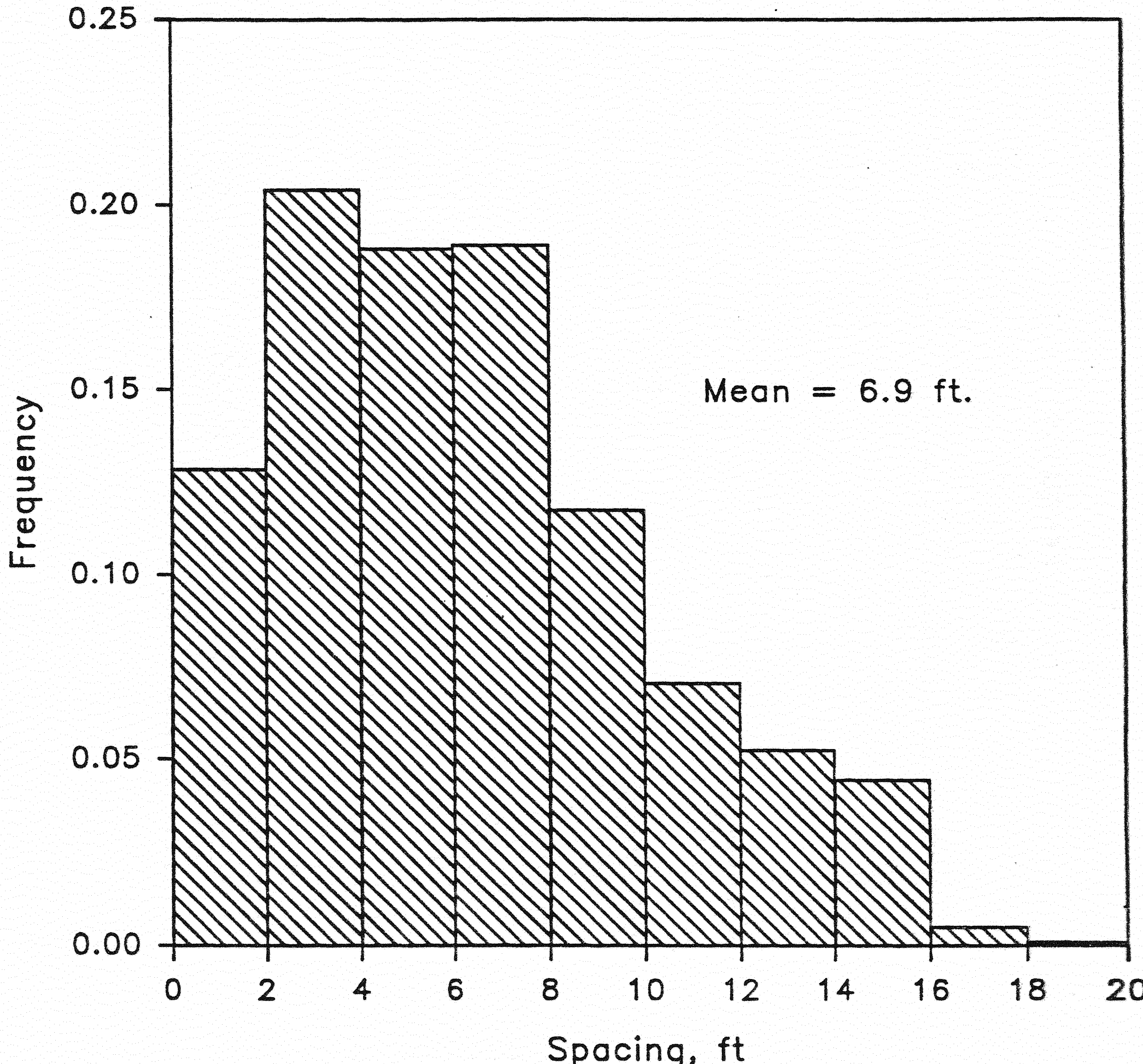


Figure 20. Histogram for Project 2, Northbound, Second Survey

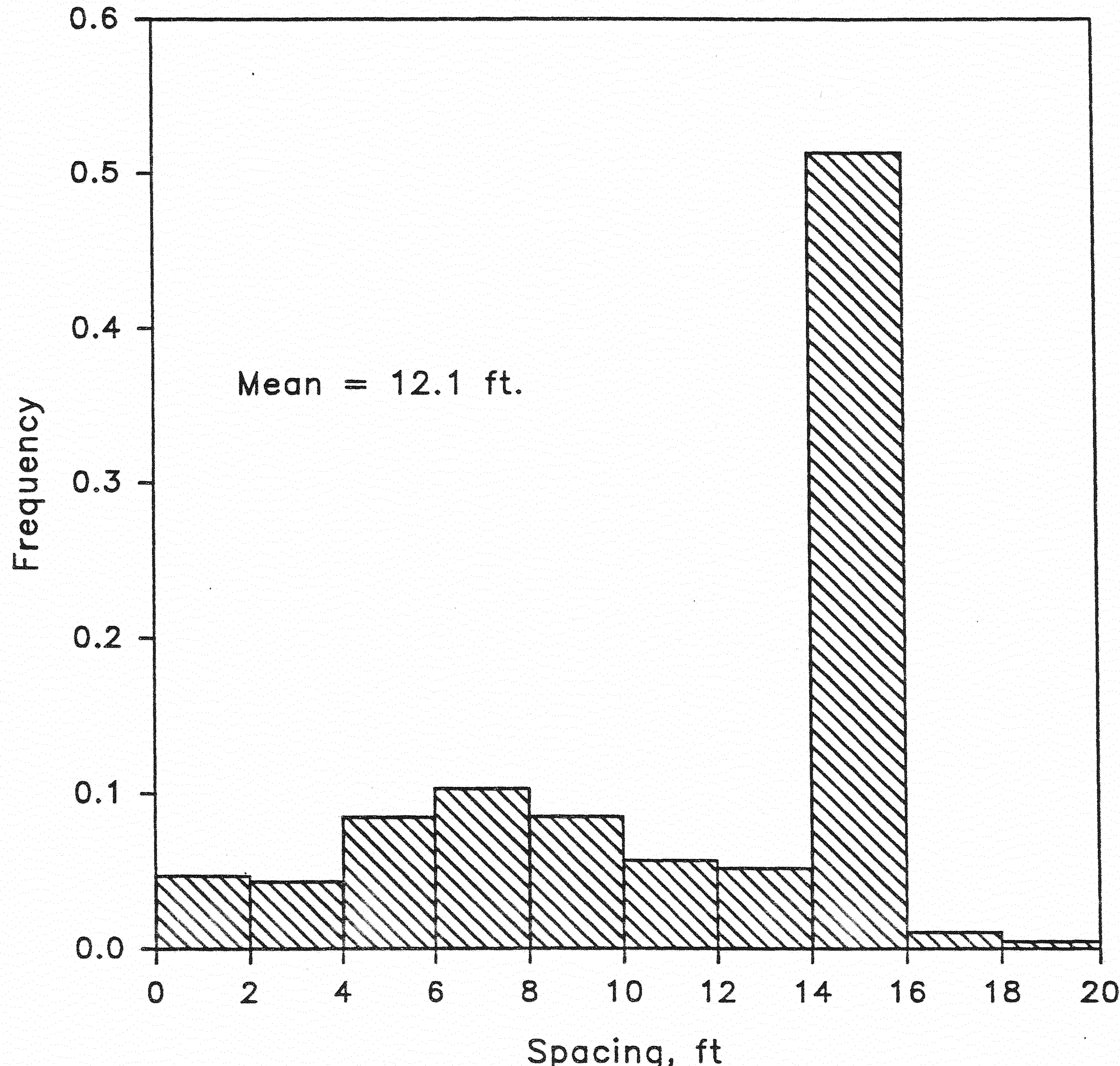


Figure 21. Histogram for Project 2, Southbound, First Survey

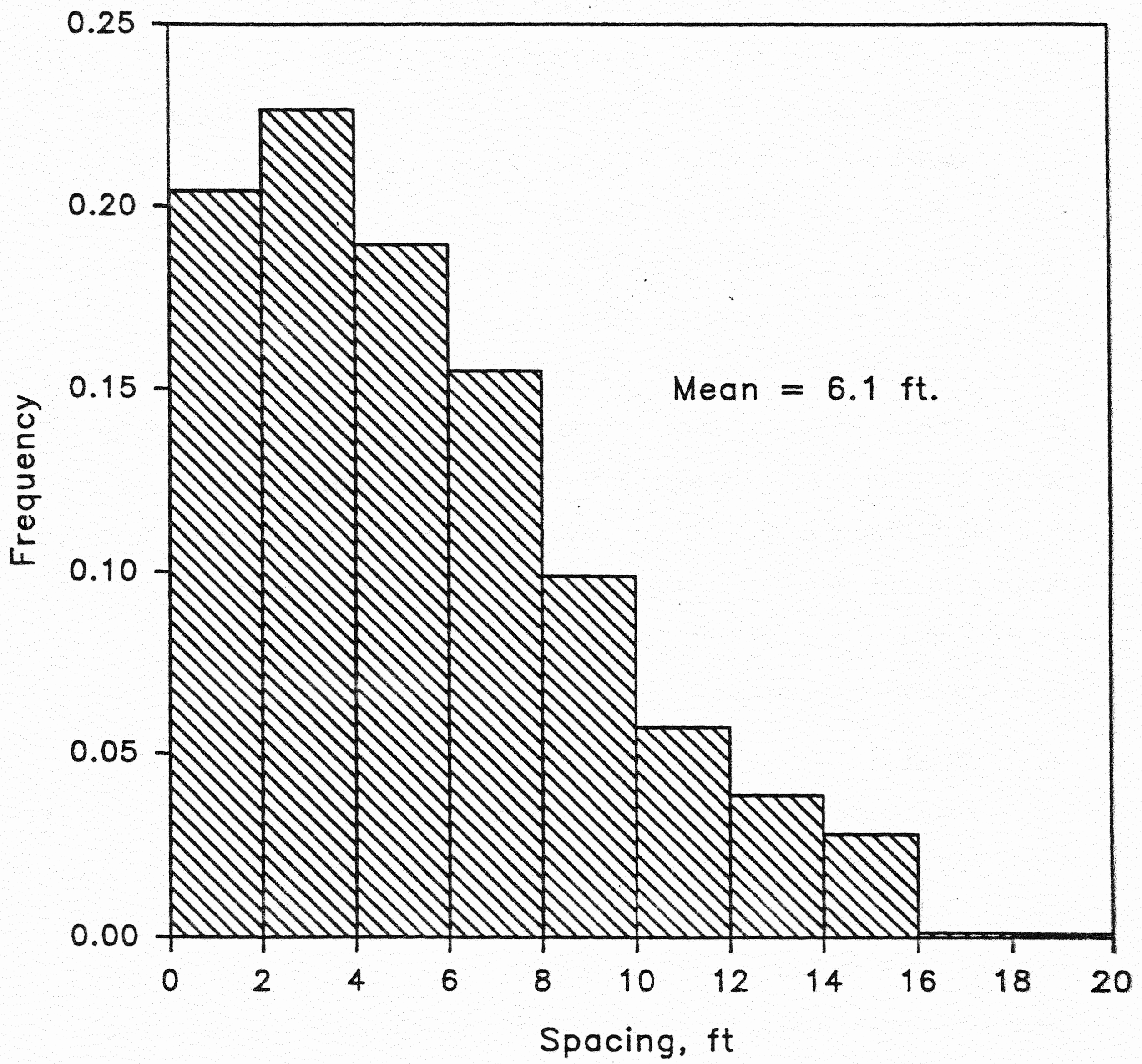


Figure 22. Histogram for Project 2, Southbound, Second Survey

was used for these comparisons. Some caution should be exercised in applying the results of this statistical comparison, since the t-test assumes normally distributed data and the present data are somewhat skewed toward lower values.

Comparing the behaviors of CRCP with coated and uncoated steel at a more subjective level, it should be noted that the range of spacings is almost identical and that the spacing with the highest frequency is always in the 2- to 4-ft interval. The age of the pavement should also be considered when conclusions are drawn from the data. It is likely tha cracking, especially for Project 2, has not stabilized. Finally, the comparison between north- and southbound lanes of Project 1 is further clouded by the fact that the steel percentage in the northbound lanes is less than in the southbound lanes.

It should also be remembered that, in the field, the reinforcement is not the only variable. Factors such as quality of subbase and subgrade, concrete strength, air temperature, air content, and slump vary over significant ranges. In the following subsections, the relationship of position along the length of the pavement, concrete 28-day compressive strength, air temperature at time of placing, air content, and slump to crack spacing are examined. Values for crack spacing are based on the second survey. Values for the control variables are taken from inspectors' notebooks.

3.1.1 Position

Average crack spacings were calculated for 1000-ft sections over the entire length of the pavements and plotted versus distance from a terminal point. The effect of distance from terminal points can be seen in Figures 23 through 26. Away from terminal points, crack spacing is relatively uniform. The very high value seen in Figure 26 away from the ends of the pavement is in the vicinity of a bridge where the pavement was also terminated. Near terminal points, crack spacing increases significantly. Such behavior near terminal points is due to a lack of fixity at these points and has been observed by others [4,13,21].

It can also be seen that crack spacing is more uniform for Project 1 than for Project 2. This can be attributed to the different ages of the pavements. Additional cracking between widely spaced cracks on Project 2 will occur with time to make the spacing more uniform. It is likely that cracking on Project 1 has nearly stabilized.

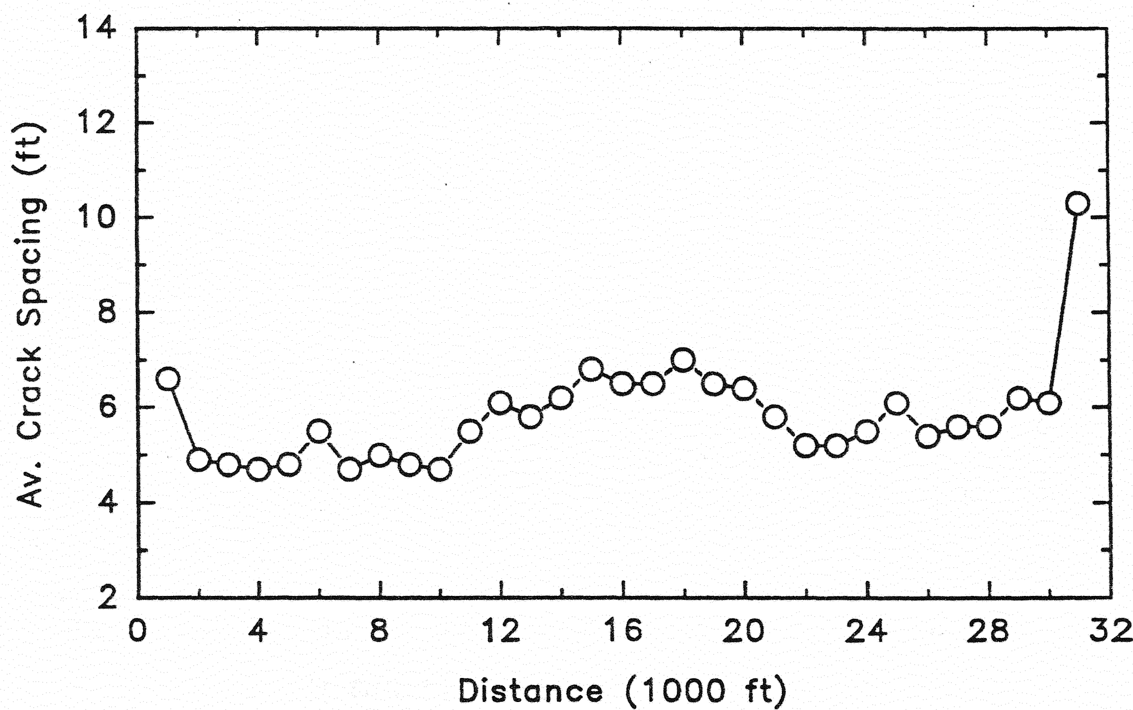


Figure 23. Effect of Distance From Terminal Point for Project 1, Northbound

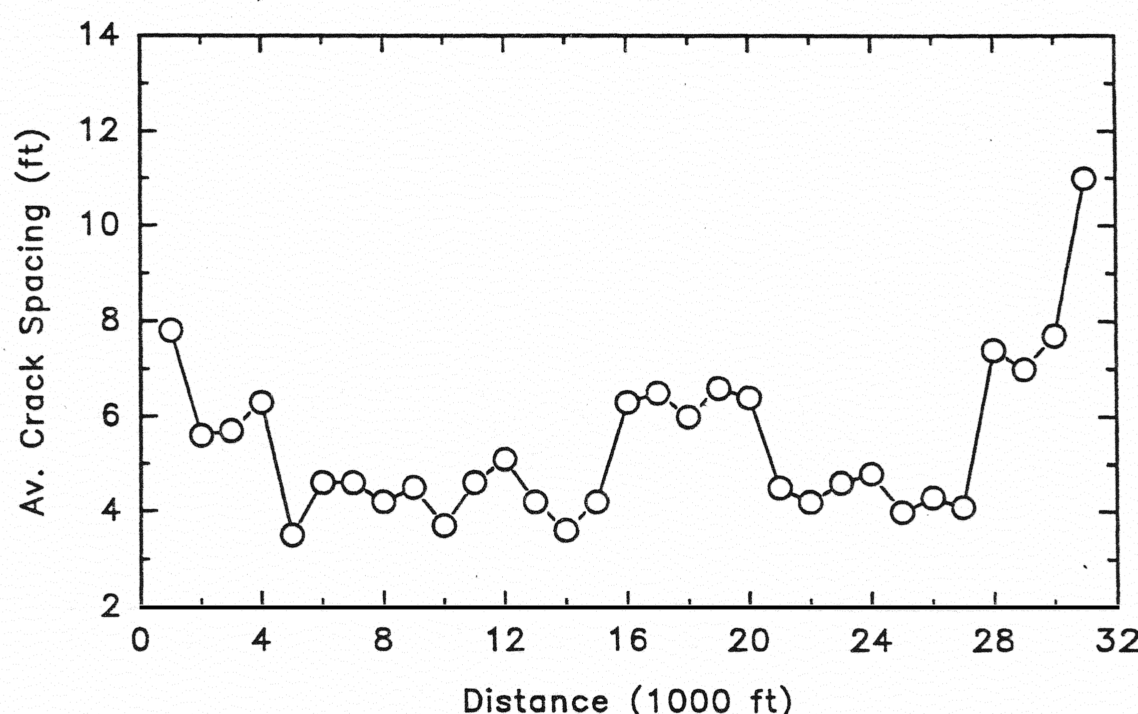


Figure 24. Effect of Distance From Terminal Point for Project 1, Southbound

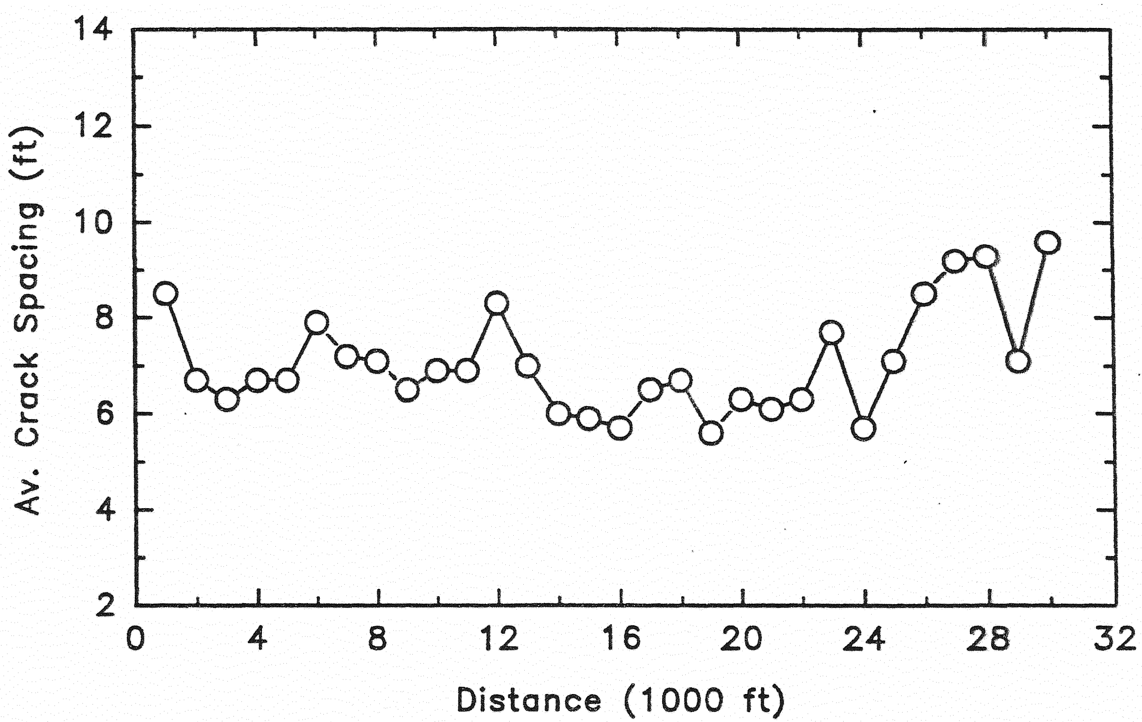


Figure 25. Effect of Distance From Terminal Point for Project 2, Northbound

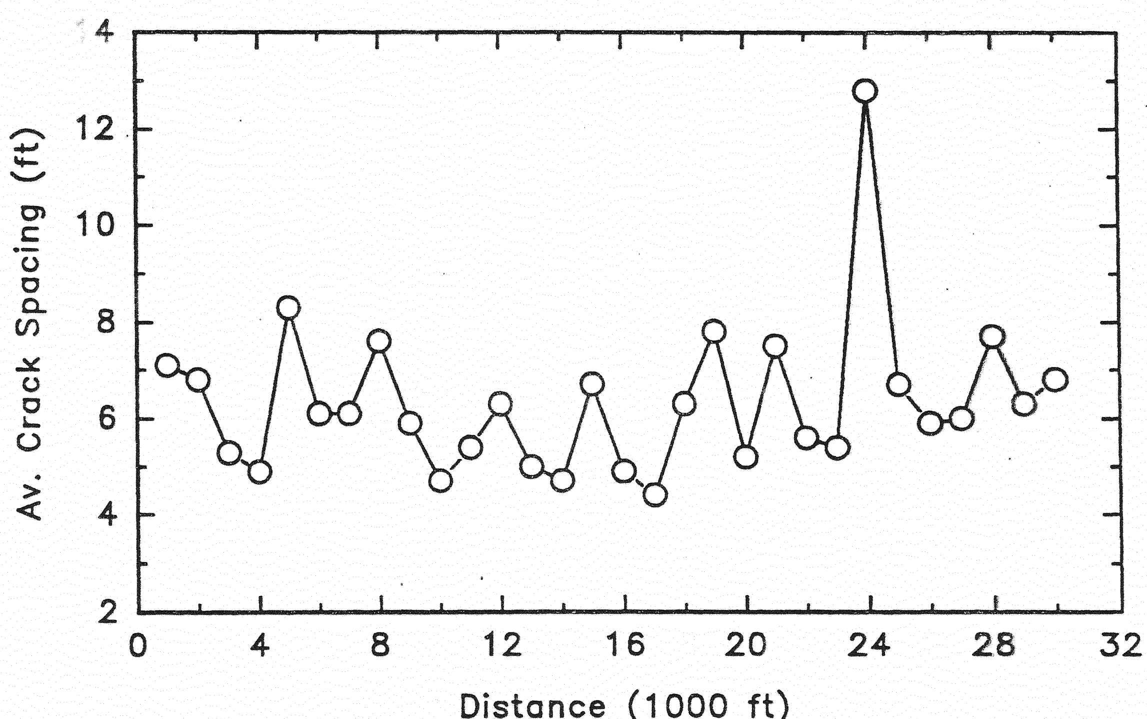


Figure 26. Effect of Distance From Terminal Point for Project 2, Southbound

3.1.2 Concrete Strength

Average crack spacing versus concrete 28-day compressive strength is plotted in Figures 27 through 30. All data are plotted to the same vertical scale. Regression lines and 95% confidence limits are shown with the data. Terminal points are indicated by circles and are not included in the regression analyses. Considering the amount of scatter in the data and the fact that two of the regression lines have a positive slope and two have a negative slope, it must be concluded that the data do not provide evidence of a relationship between concrete strength and crack spacing. This conclusion is contrary to the findings of an earlier Texas study [49] in which it was concluded that an increase in concrete strength led to an increase in crack spacing.

3.1.3 Air Temperature

Average crack spacing versus air temperature at the time of placement is plotted in Figures 31 through 34. The format of the plot is the same as for concrete strength. In three of the four plots, average crack spacing decreases with increasing air temperature. This behavior is in agreement with an earlier Texas study [49]. In the single plot for which crack spacing increases with increasing temperature, temperatures were consistently higher than for the other three cases. The data from the Texas report covered a 60 to 90°F temperature range, similar to the data presented in Figures 31 through 33. This behavior suggests that the influence of air temperature on crack spacing cannot be modeled linearly.

3.1.4 Air Content

Average crack spacing versus concrete air content is plotted in Figures 35 through 38 in the same format as the preceding figures. The regression lines have positive slopes in all four plots, indicating a slight increase in average crack spacing with increasing air content. Independent confirmation of this behavior was not found.

3.1.5 Slump

Average crack spacing versus slump is plotted in Figures 39 through 42. The slopes of the regression lines in three of the plots are negative and in one is positive. Considering the

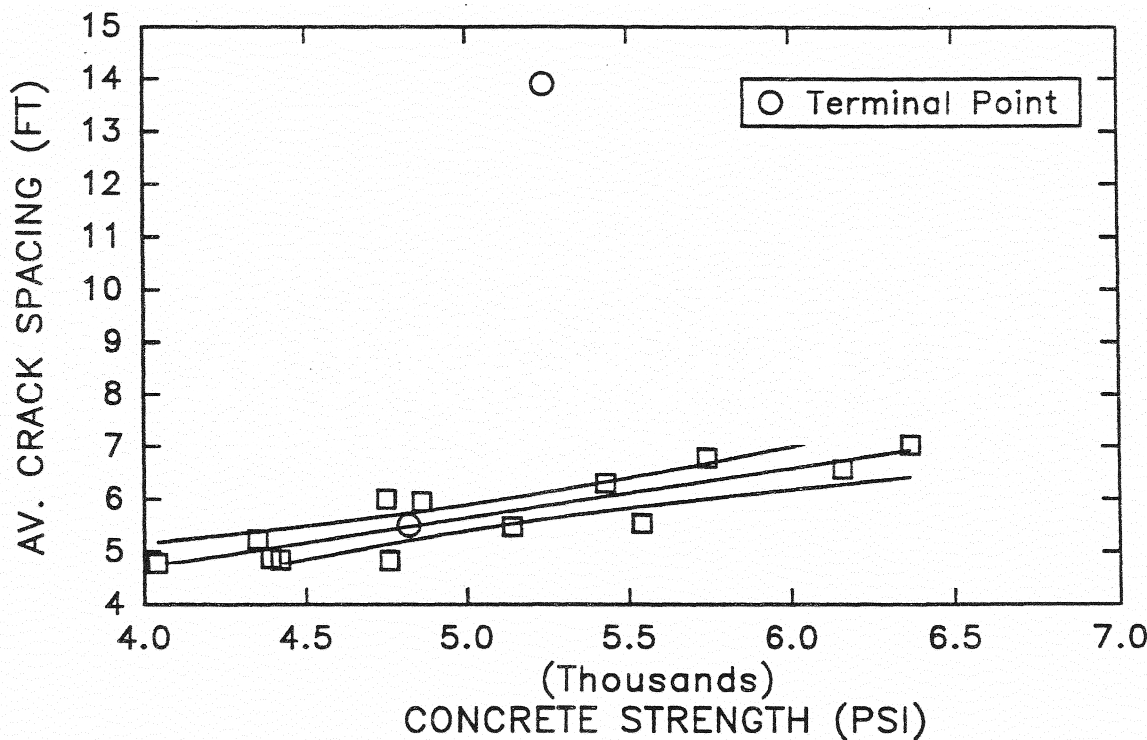


Figure 27. Effect of Concrete Strength for Project 1, Northbound

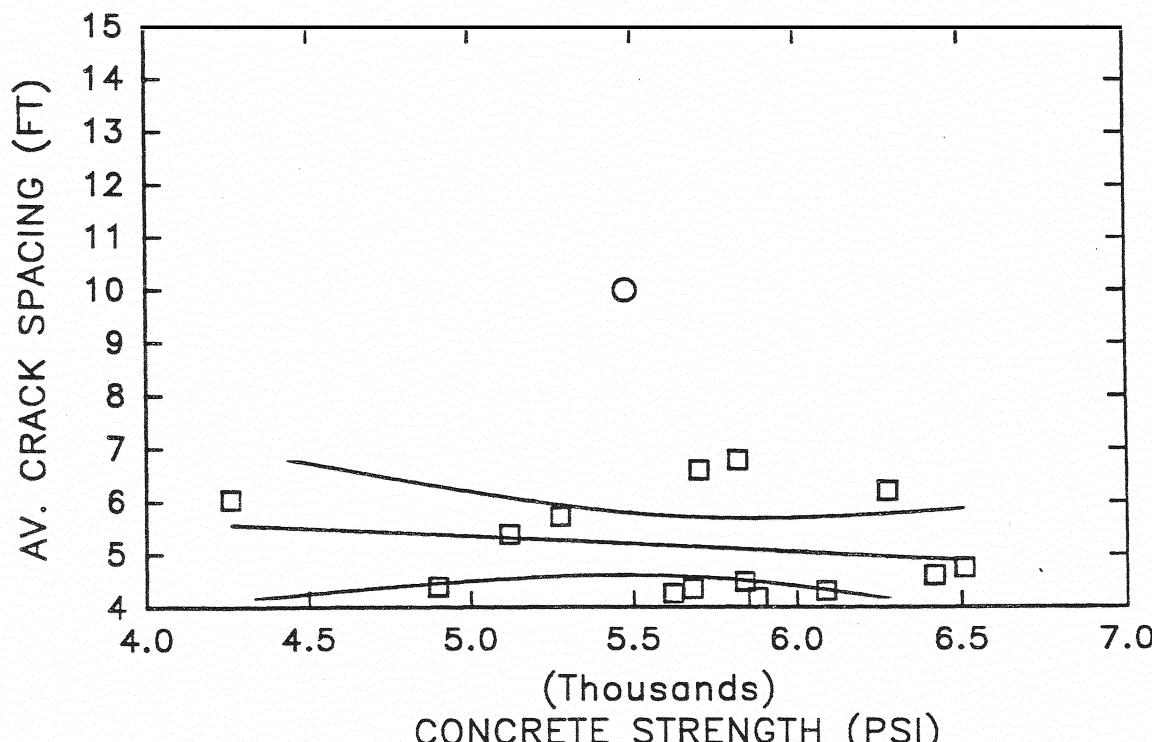


Figure 28. Effect of Concrete Strength for Project 1, Southbound

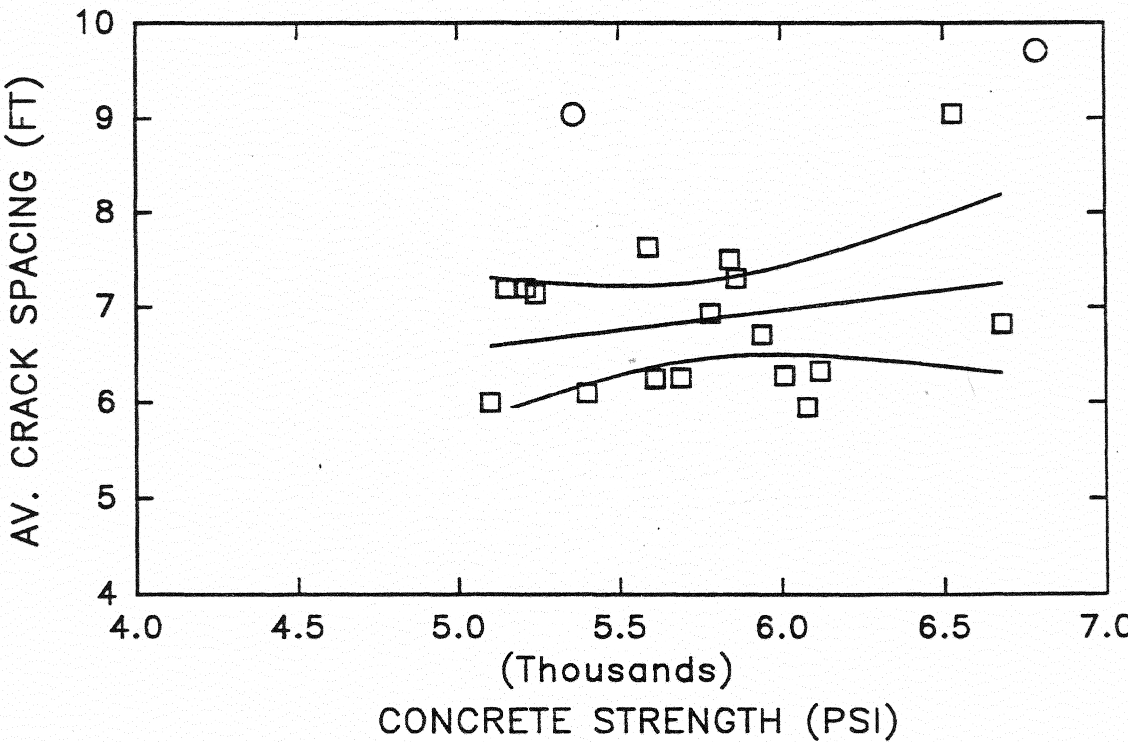


Figure 29. Effect of Concrete Strength for Project 2, Northbound

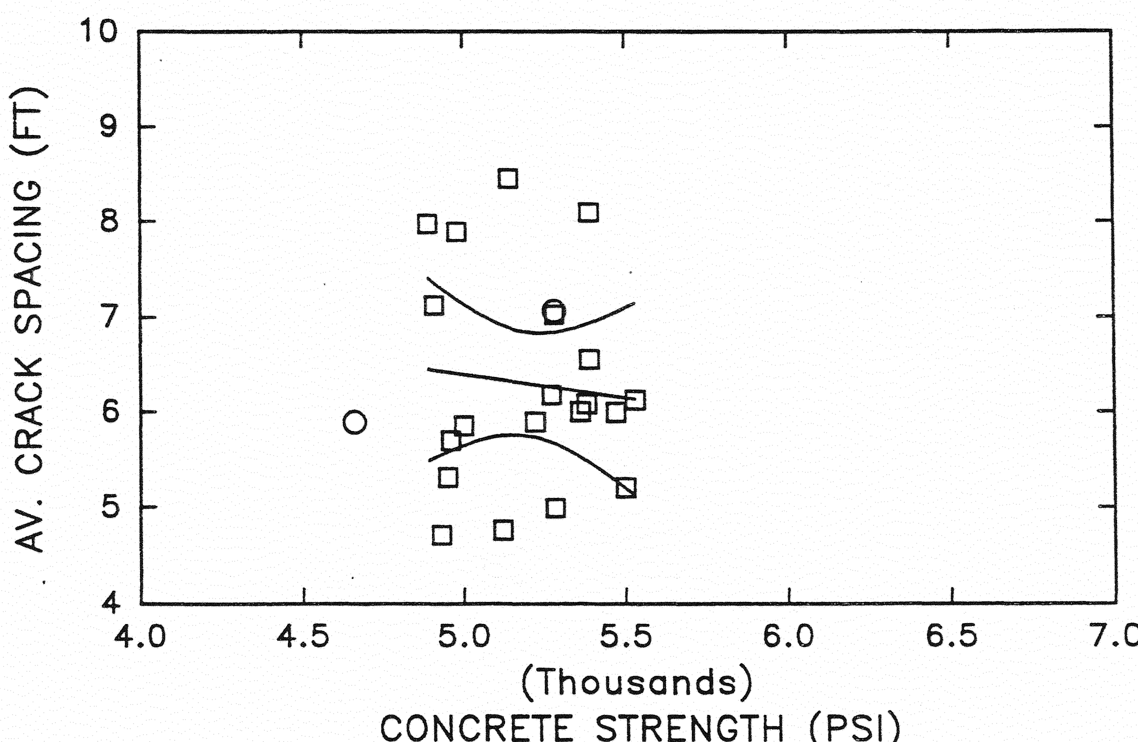


Figure 30. Effect of Concrete Strength for Project 2, Southbound

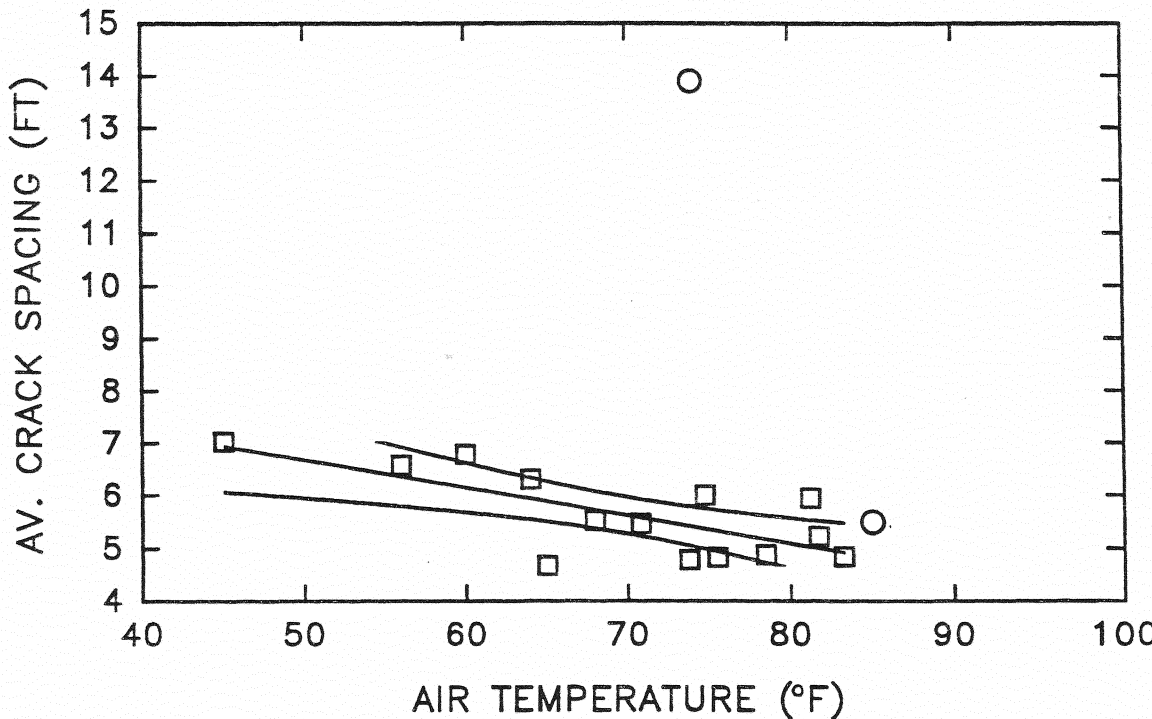


Figure 31. Effect of Air Temperature for Project 1, Northbound

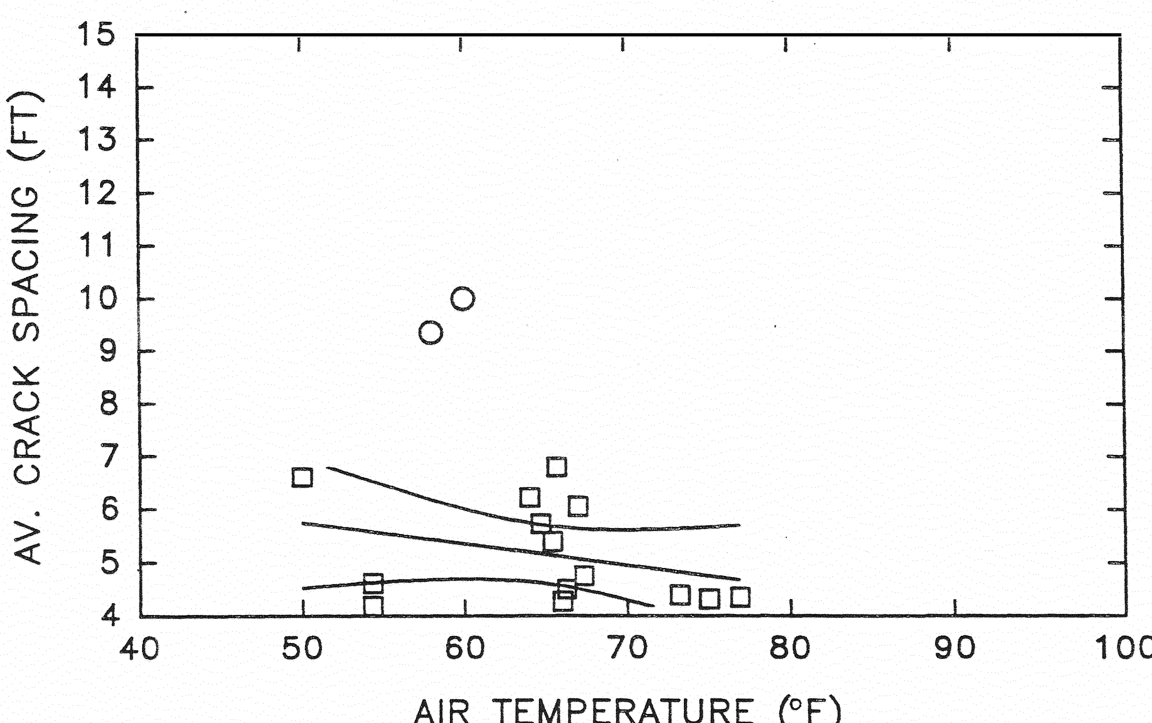


Figure 32. Effect of Air Temperature for Project 1, Southbound

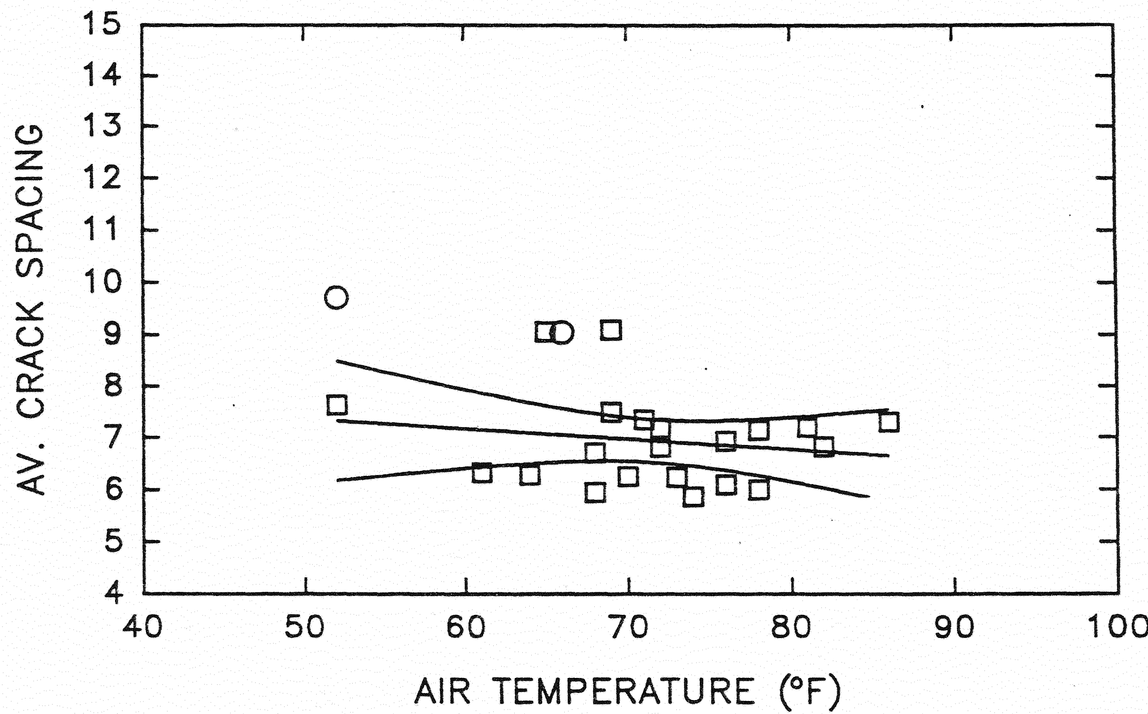


Figure 33. Effect of Air Temperature for Project 2, Northbound

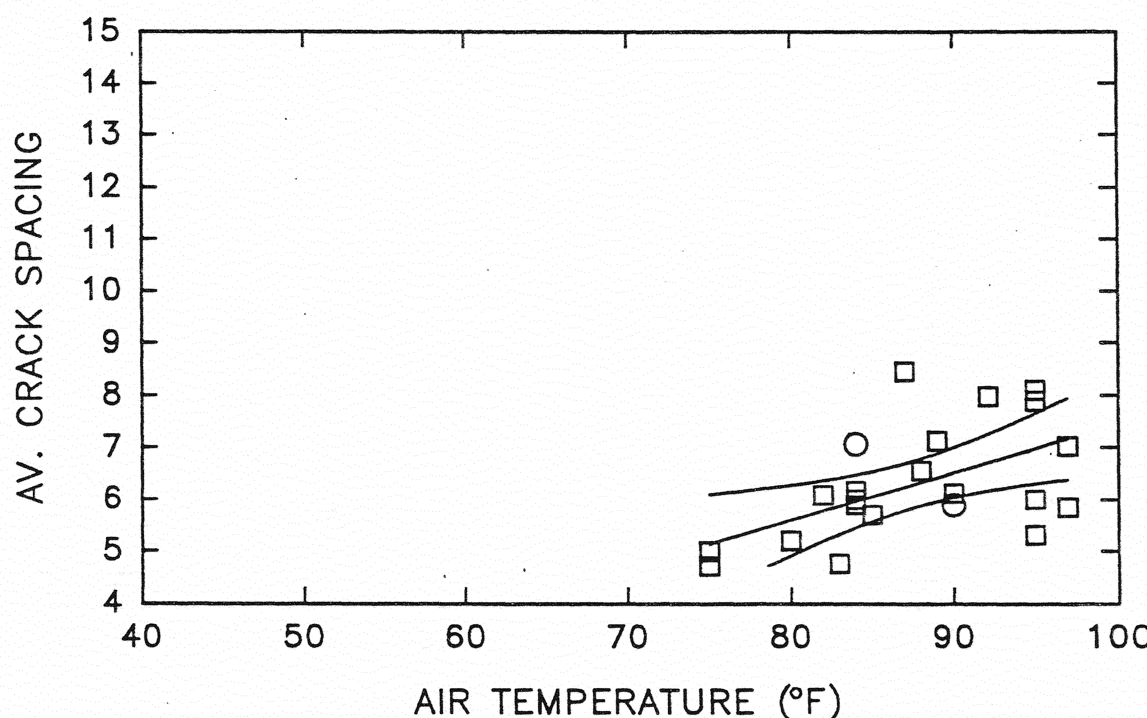


Figure 34. Effect of Air Temperature for Project 2, Southbound

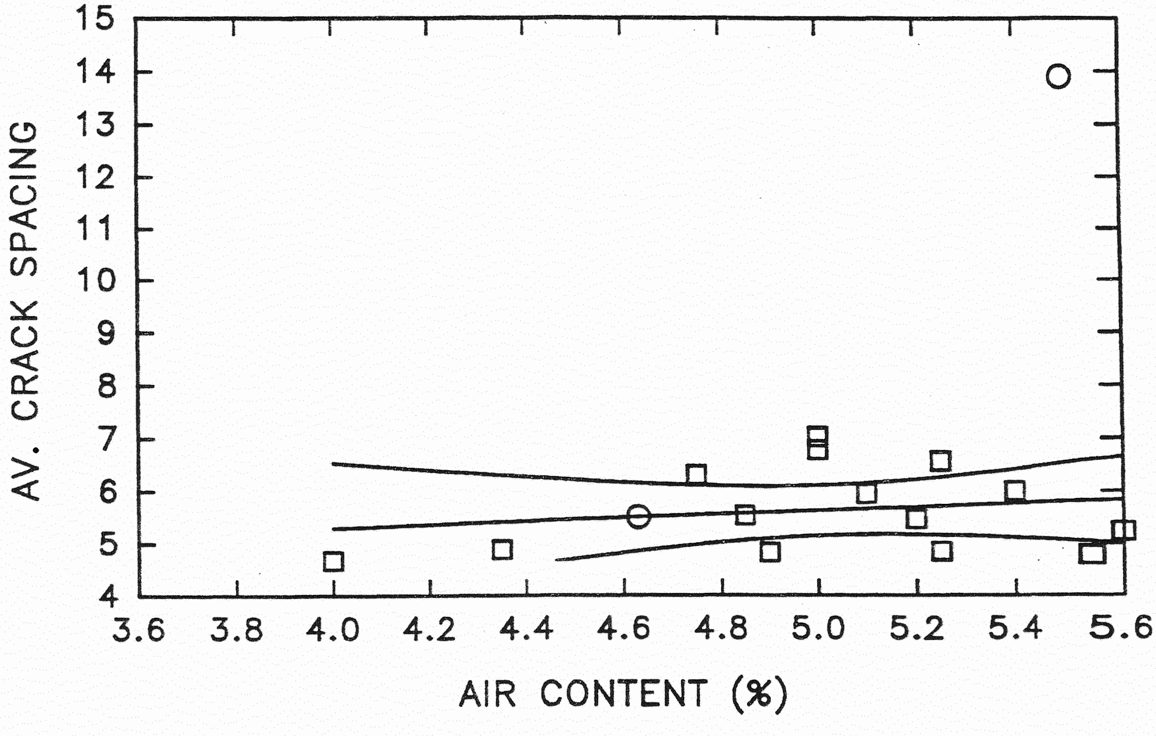


Figure 35. Effect of Air Content for Project 1, Northbound

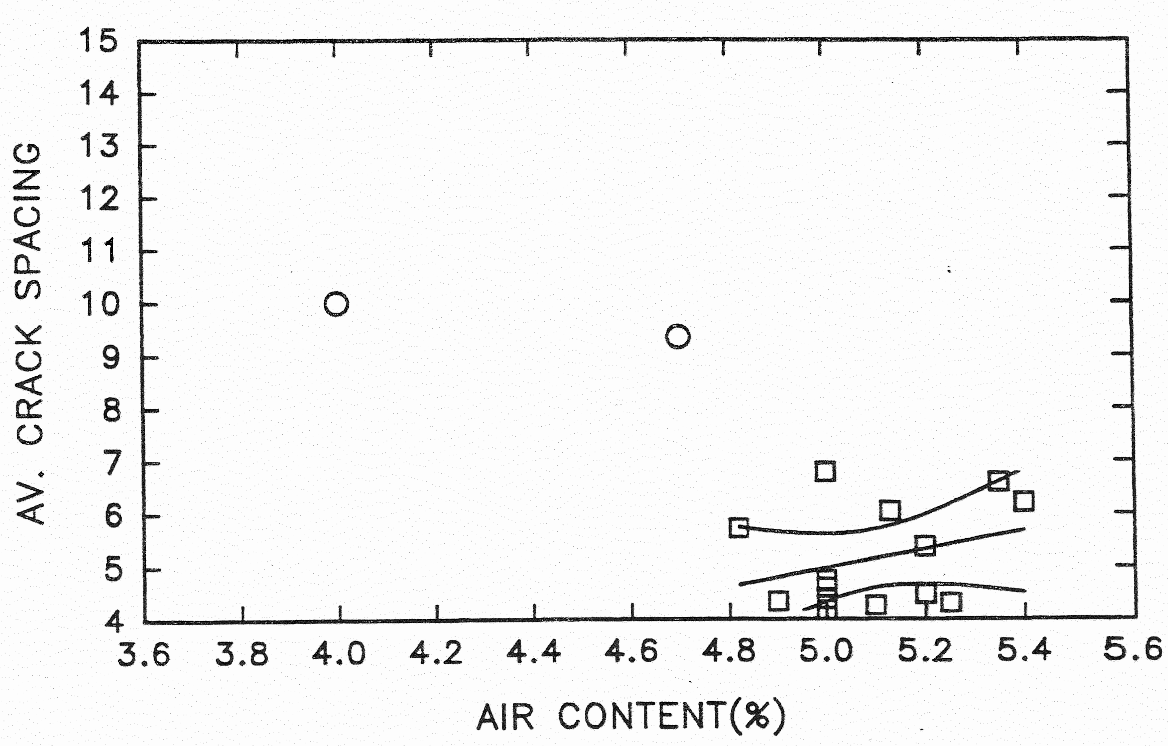


Figure 36. Effect of Air Content for Project 1, Southbound

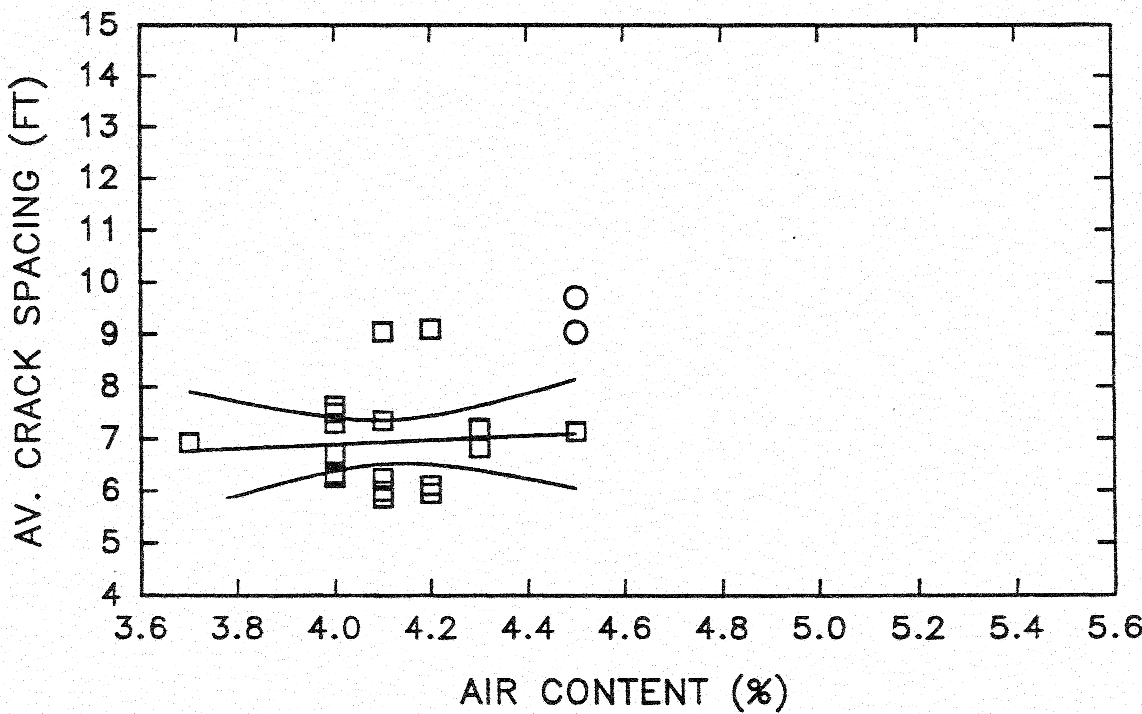


Figure 37. Effect of Air Content for Project 2, Northbound

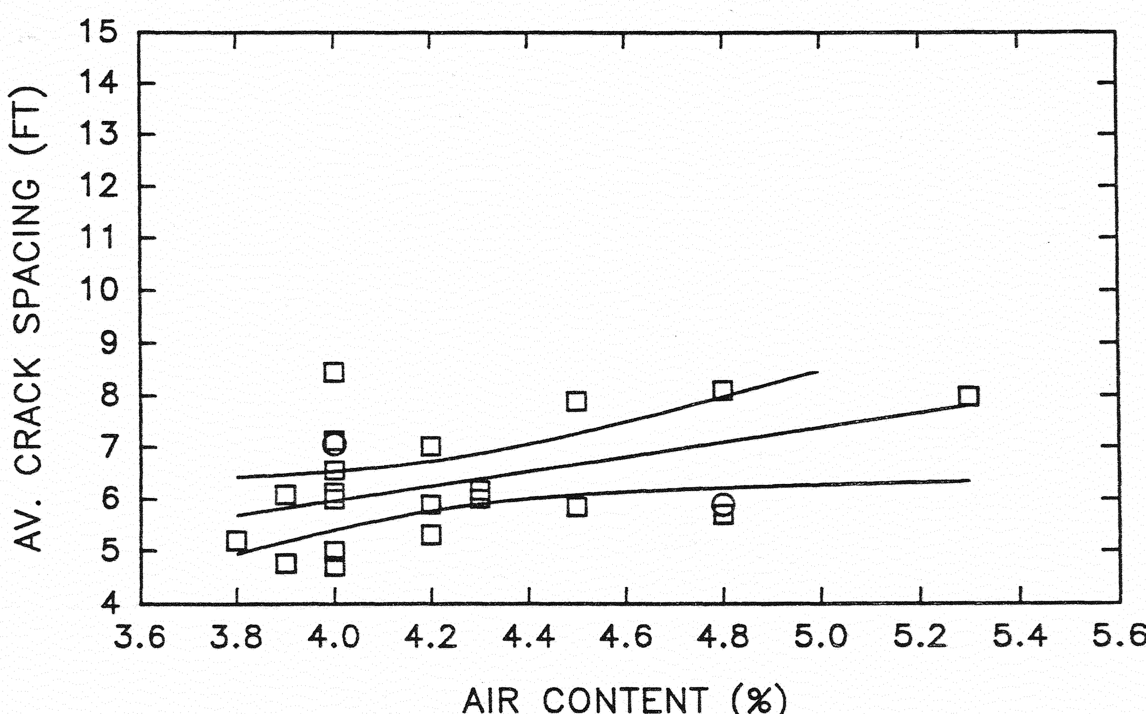


Figure 38. Effect of Air Content for Project 2, Southbound

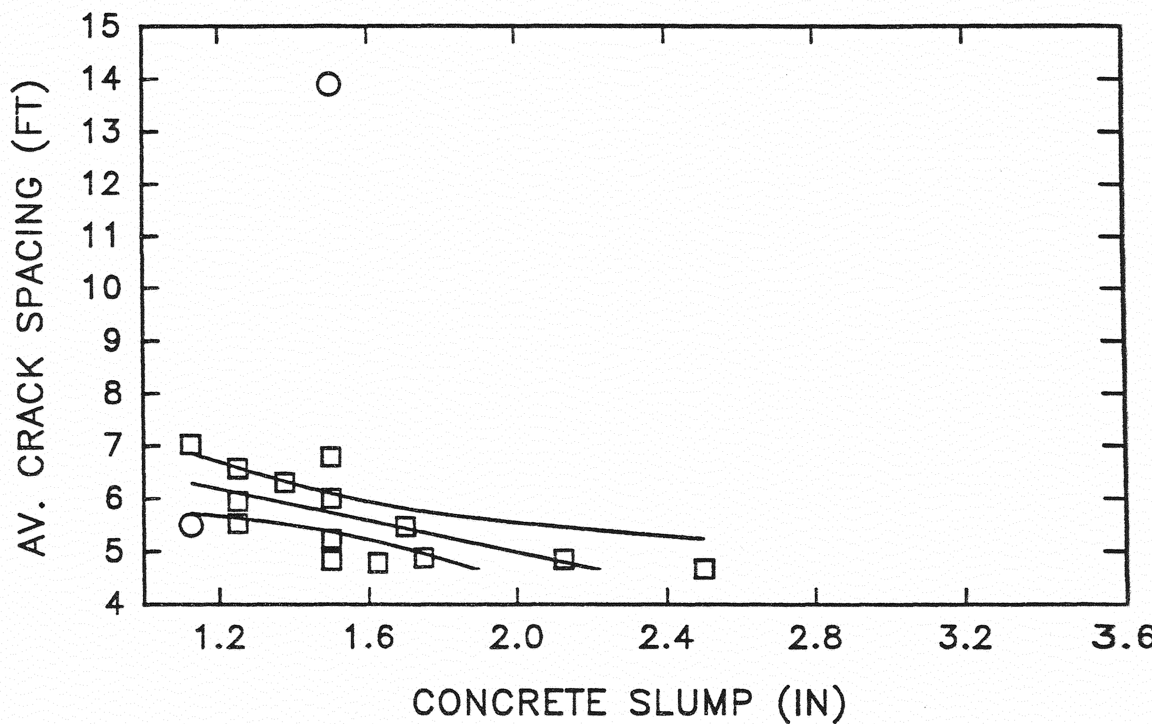


Figure 39. Effect of Slump for Project 1, Northbound

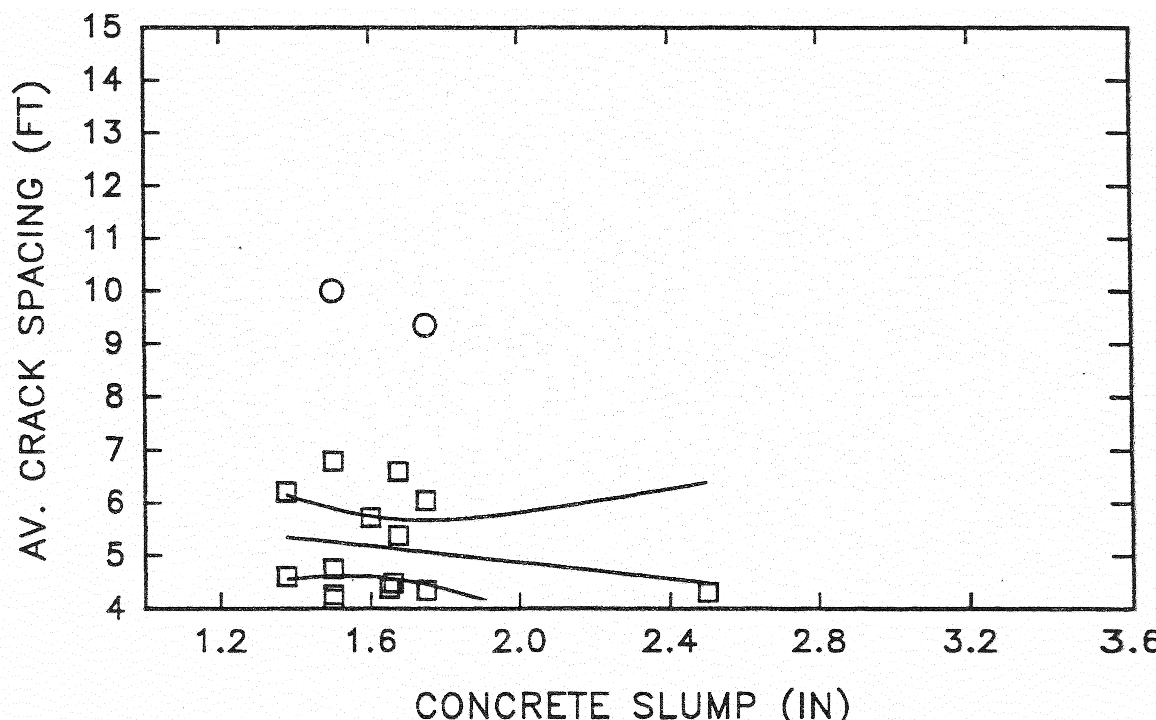


Figure 40. Effect of Slump for Project 1, Southbound

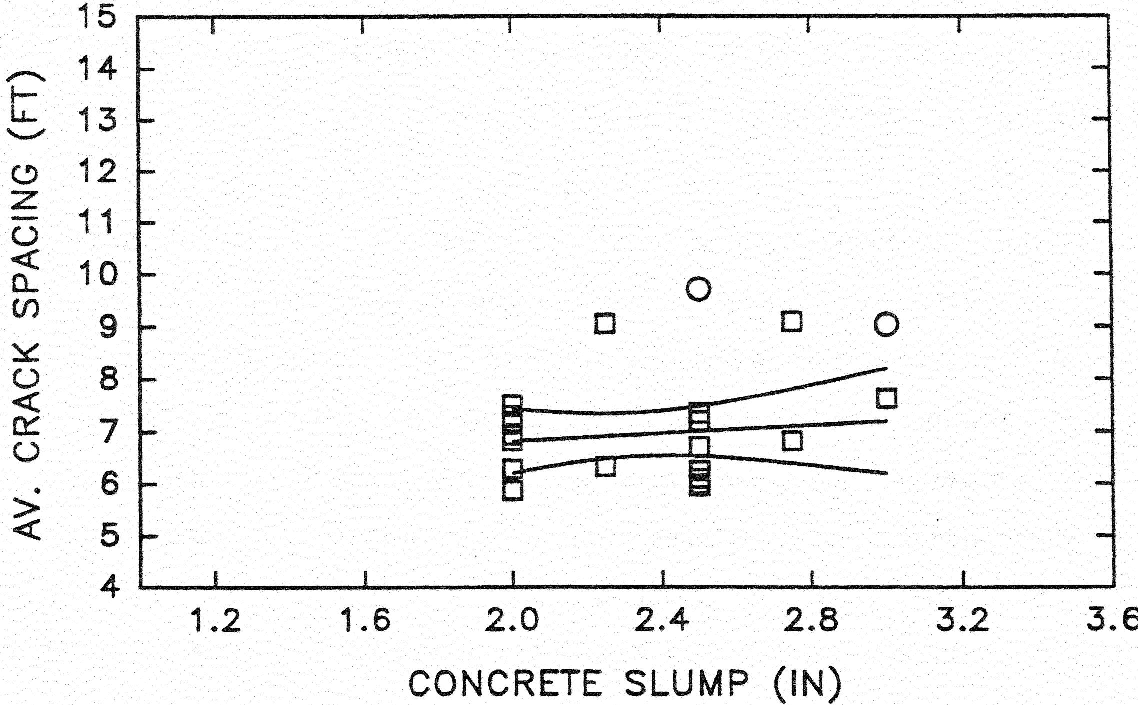


Figure 41. Effect of Slump for Project 2, Northbound

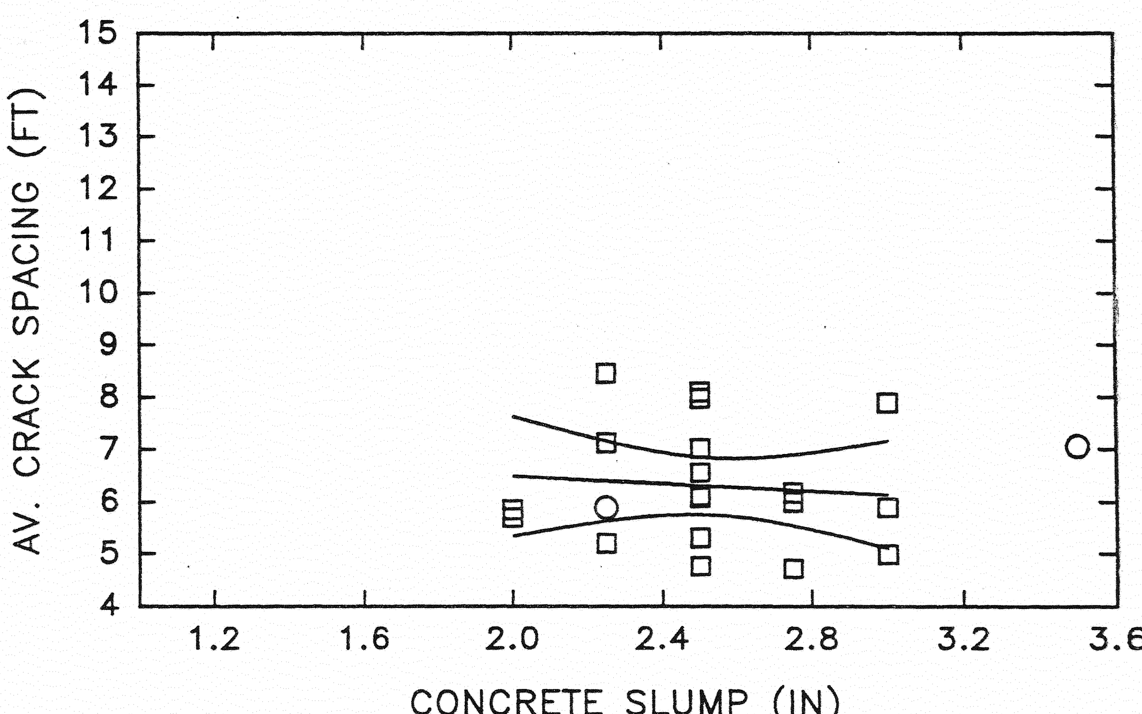


Figure 42. Effect of Slump for Project 2, Southbound

scatter in the data along with the differences in the regression lines, it must be concluded that a relationship between crack spacing and slump is not indicated.

3.2 Summary

Transverse cracks continue to form in all pavements, but appear to be stabilizing on Project 1. If the present trend continues, the average crack spacing will stabilize at between 4 and 6 ft. The average crack spacing on pavements constructed with epoxy-coated steel is less than on pavements constructed with uncoated steel, but the difference is small.

The effect of concrete strength, air temperature, air content, and slump on crack spacing is not well defined. Of the variables listed, air temperature at the time of placement and air content appear to have the greatest impact on crack spacing. However, it should be remembered that several of these variables may be changing at the same time, which could conceal relationships that may be present.

CHAPTER 4

INSTRUMENTATION INSTALLED FOR FUTURE RESEARCH

4.1 Half Cell Test Sites

In order to test for corrosion of the reinforcing steel inside the pavement, instrumentation was installed to measure the half cell potentials. The standard test method described in the American society for testing and materials (ASTM) Specification C876-80 was followed for this installation.

The circuit for this test is shown in the Figure 43. The voltmeter measures the potential difference between the copper-copper sulfate half-cell (shown in Figure 44) and the steel reinforcement. If half-cell potentials are numerically less than -0.20 v, there is a greater than 90% probability that no reinforcing steel corrosion is occurring in that area. If half-cell potentials range from -0.20 to -0.35 v, corrosion activity in that area is uncertain. If half-cell potentials are greater than -0.35 v, there is a greater than 90% probability that reinforcing steel corrosion is occurring in that area.

One half-cell site each is installed on the north- and southbound lanes of Project 2. The sites are at station 975+100 on the northbound lanes and 805+160 on the southbound lanes. Station numbers are stamped in the pavement at 500-ft intervals, so sites can be easily located relative to these stamps. No half-cell sites are installed on Project 1.

Cadwelding was used to connect the copper leadwires to the reinforcing steel. Figures 45 and 46 show copper wires welded to reinforcing bars. The insulated copper wires are extended underground to a small concrete box. Figure 47 shows the wires concealed in the ground. Nine copper wires are welded to the coated steel on the northbound lanes and three wires are welded to the uncoated steel on the southbound lanes. More lead wires are welded on the northbound lanes because it was not expected that electrical continuity would exist between epoxy-coated bars. Figure 48 shows the location of the concrete boxes with respect to the pavement and demonstrates the numbering scheme used to identify the individual leadwires.

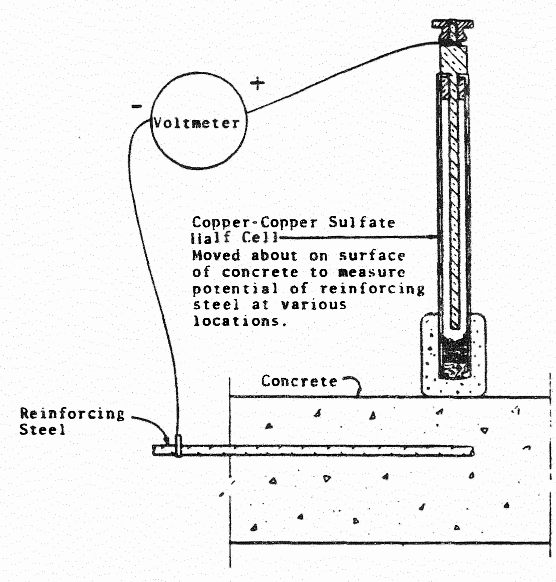


Figure 43. Copper-Copper Sulfate Half-Cell Circuitry [1]

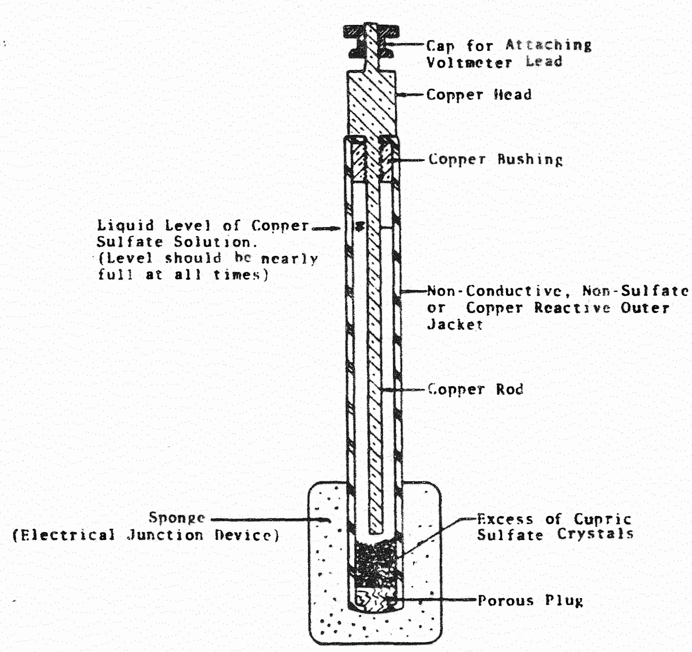


Figure 44. Sectional View of a Copper-Copper Sulfate Half-Cell [1]

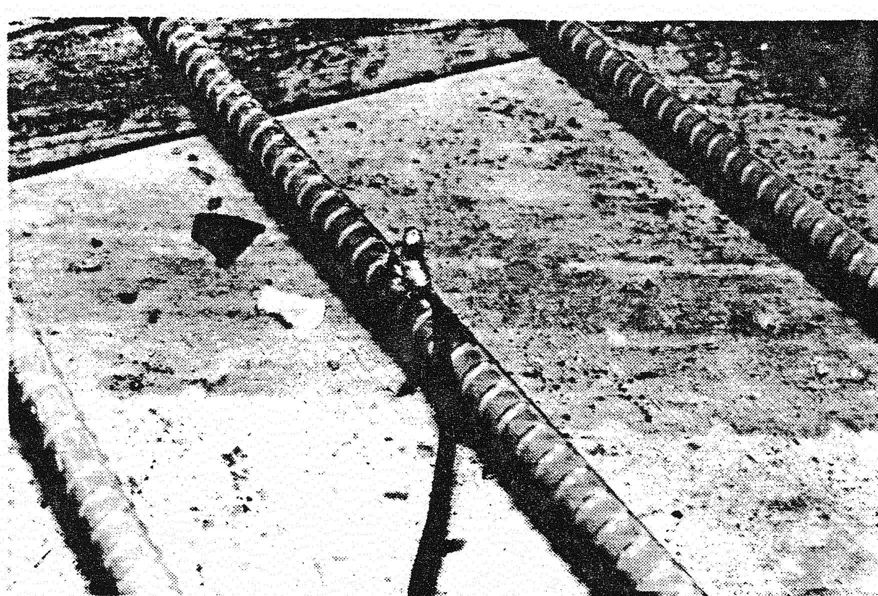


Figure 45. Copper Wires Welded to Reinforcing Bar

Figure 46. Copper Wires Connected to Reinforcing Bars

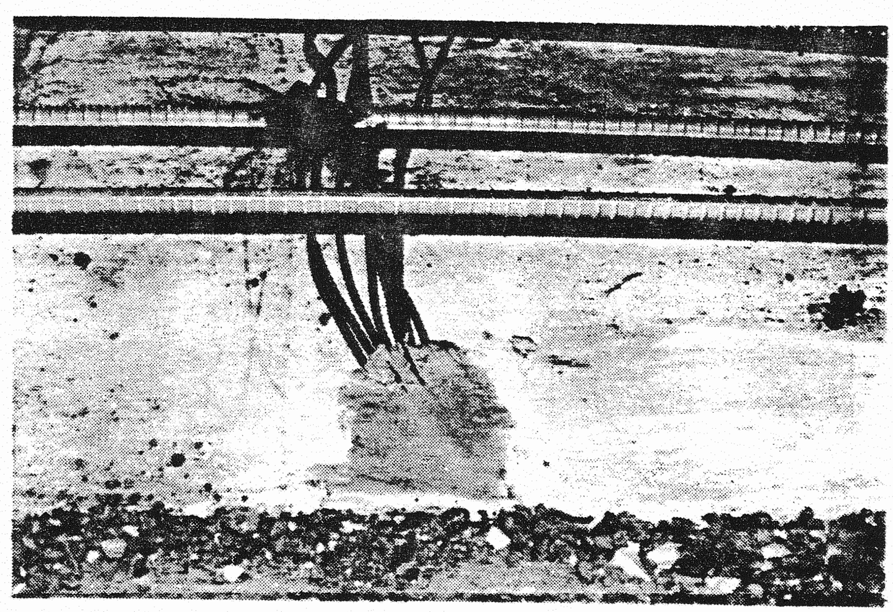


Figure 47. Copper Wires Carried Through the Ground

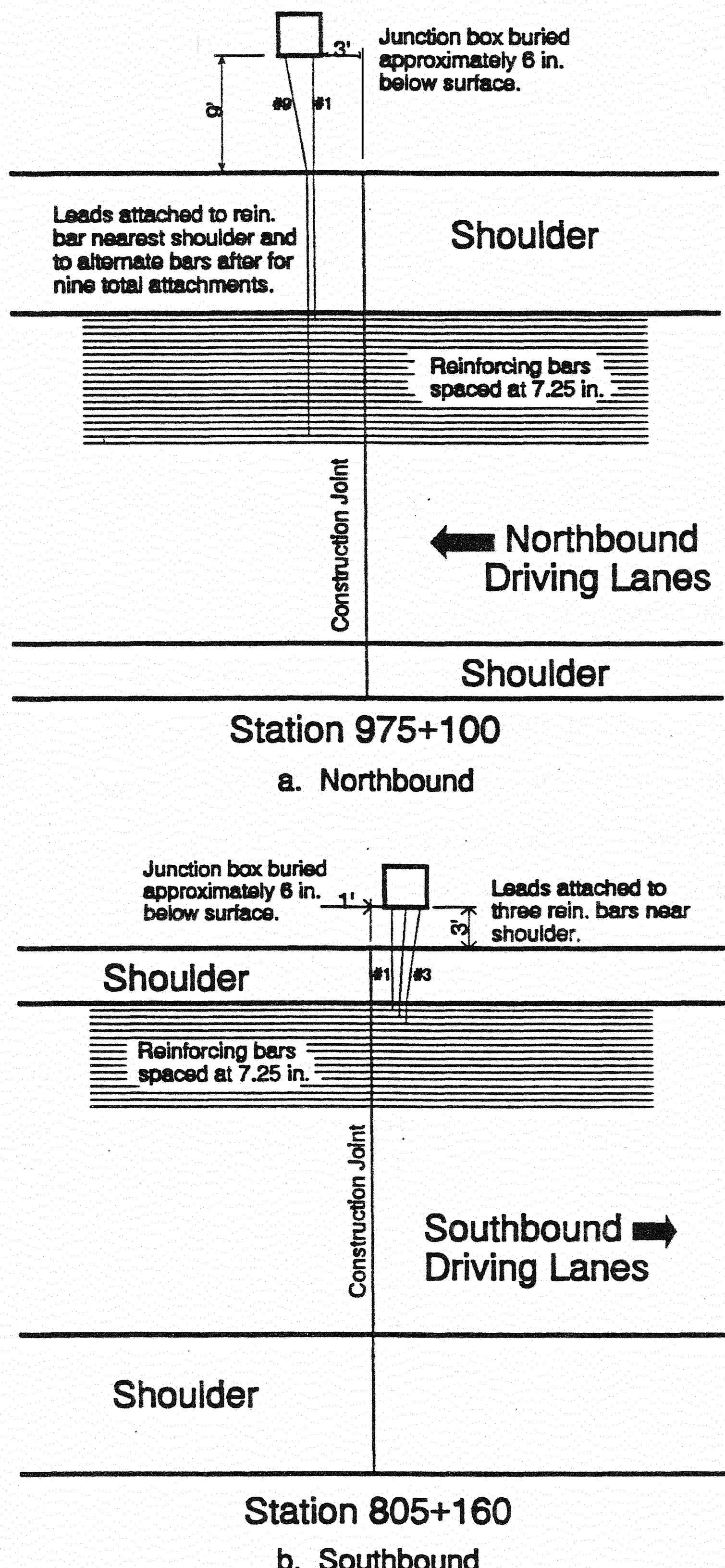


Figure 48. Locations of Half-Cell Sites

Half-cell potentials were measured on both lanes of Project 2, before opening to traffic. The potentials for northbound lanes varied from -0.169 to -0.274 v and for southbound lanes varied from -0.066 to -0.190 v. These measurements indicate that corrosion activity was nonexistent to slight at the time of measurements.

In addition to measuring half-cell potentials, electrical continuity between coated reinforcing bars on the northbound lanes was examined. It was found that four of the bars (bars #1, 2, 6, and 7) were electrically connected. The other five bars were electrically isolated.

4.2 Gauge Points to Measure Crack Width

In order to measure changes in crack width, gauge points were installed at selected cracks on north- and southbound lanes of Project 2. These cracks were selected for monitoring immediately after pavement construction. Small holes were drilled on either side of the cracks and two brass plugs were fixed in the holes using epoxy. Gauge points were marked on the exposed surface of the plugs. A total of 6 gauge points were installed on northbound lanes and 9 gauge points on southbound lanes of Project. Figure 49 shows a typical gauge point. The distance between gauge points was measured using a Whittemore gauge. Crack width fluctuations can be assessed by comparing subsequent measurements to initial readings. No gauge points were installed on Project 1.

Crack widths were also measured from photographs taken through a comparator. Typical photographs are shown in Figure 50. The full width of the grid over the cracks is 2 mm. Crack widths as measured with the comparator were found to vary between 0.2 and 0.3 mm (0.008 and 0.012 in.).

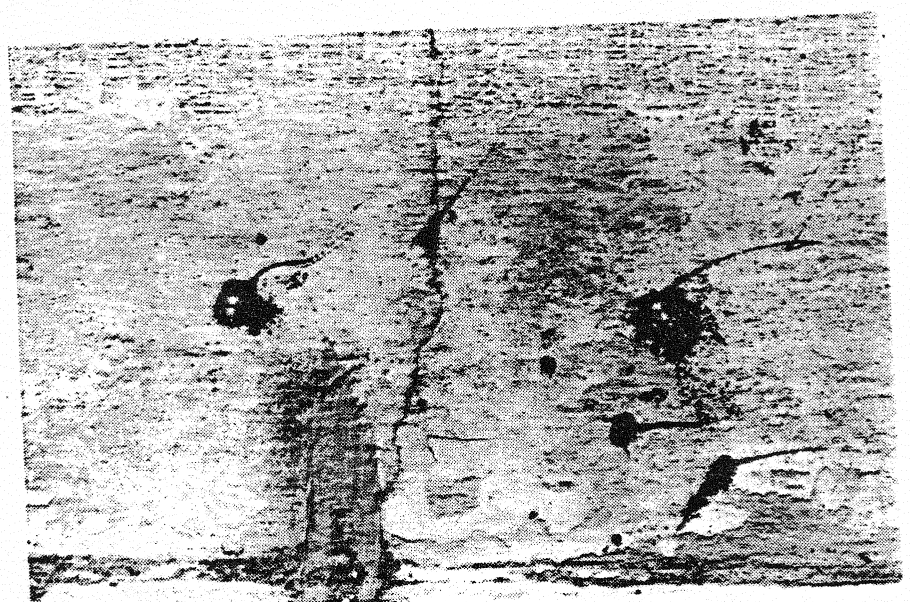


Figure 49. A Typical Gauge Point

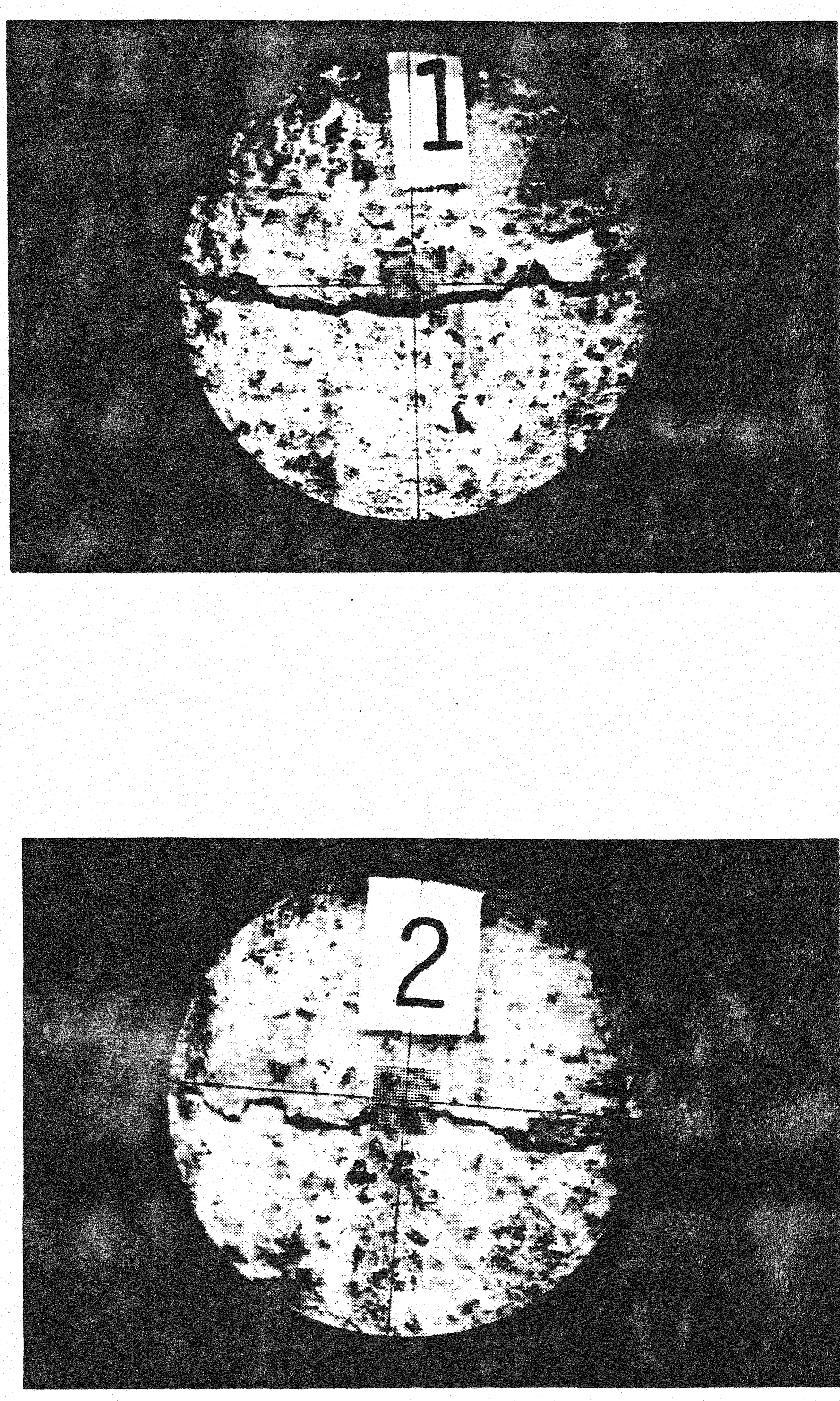


Figure 50. Cracks Photographed Using a Comparator

CHAPTER 5

LABORATORY TESTING OF TENSILE SPECIMENS

As discussed in Chapter 3, a direct comparison of pavements constructed with epoxy-coated steel to pavements constructed with uncoated steel is difficult in the field. Many factors other than steel coating, such as concrete strength and air content, may vary from pavement to pavement and affect performance. To better control these incidental factors, specimens were constructed in the laboratory to further study the influence of epoxy coating.

5.1 Description of Specimens

Drawings of the pavement specimens are provided in Figures 51 and 52. All specimens were poured simultaneously to achieve uniformity with respect to concrete properties. Concrete was poured onto a leveled sand base which was spread over a plastic sheet on a concrete floor. The intent was to provide similar subbase support and subgrade friction for all specimens.

A total of four specimens were constructed: two as shown in Figure 51 and two as shown in Figure 52. One of each pair was reinforced with epoxy-coated steel and the other with uncoated steel. Longitudinal reinforcing bars were 3/4 in. in diameter for all specimens, resulting in steel percentages of 0.61 and 0.73. One-half-in. diameter transverse bars were spaced at 44 in. along the length of the specimens in accordance with ODOT standards. The longitudinal bars rested on top of the transverse bars and were at midthickness of the slabs. All reinforcing steel was grade 60.

5.2 Test Apparatus

The load frame and measurement devices used in these tests are shown in Figure 53. The primary design criterion for the frame was that it operate under displacement control in order to achieve accurate measurement of length change due to formation of a crack. When a crack forms, the load suddenly drops. If the test was conducted under load control, the loading device would immediately try to compensate by increasing displacement. Under displace-

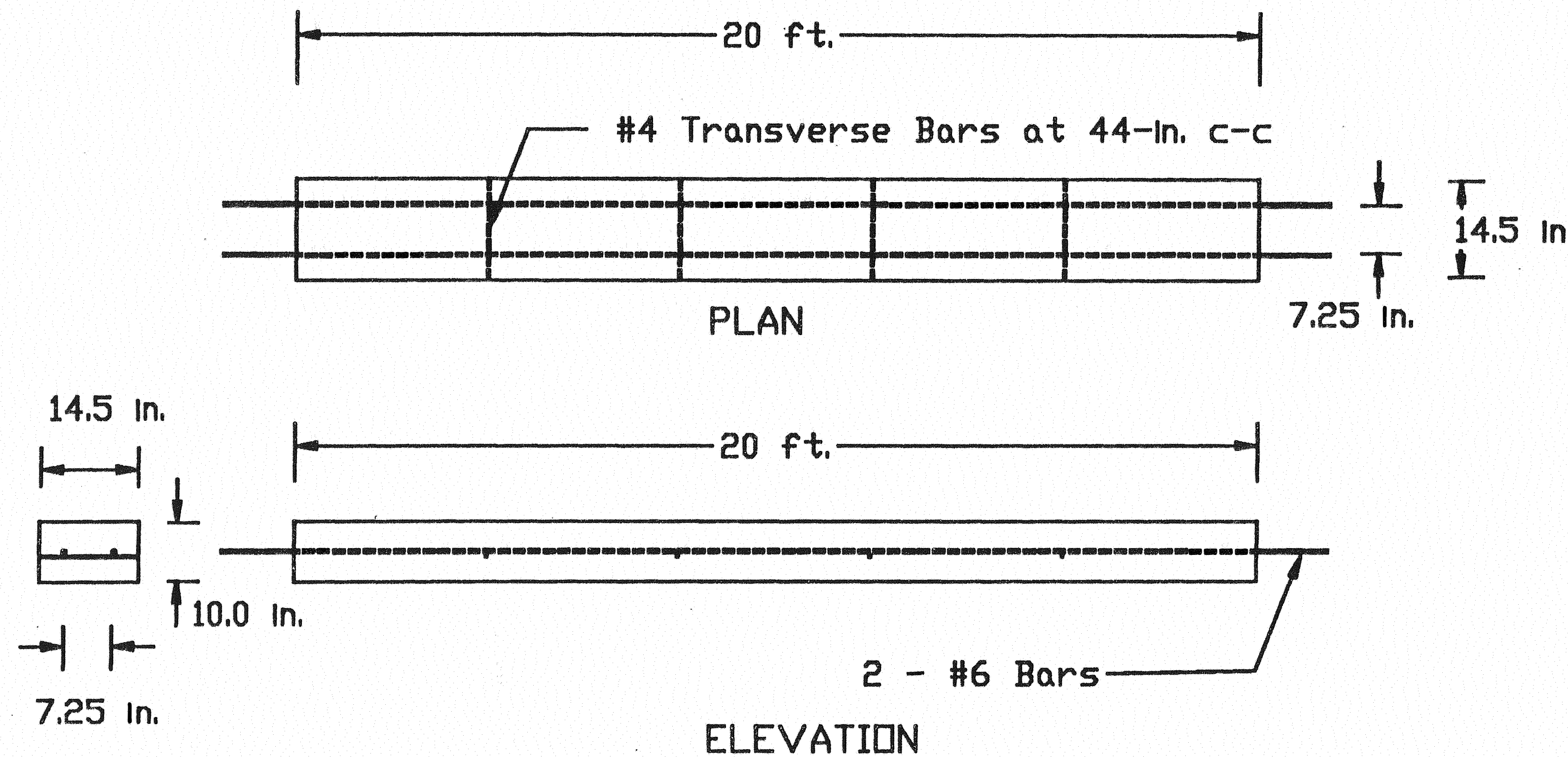


Figure 51. Slabs With 0.61% Reinforcement

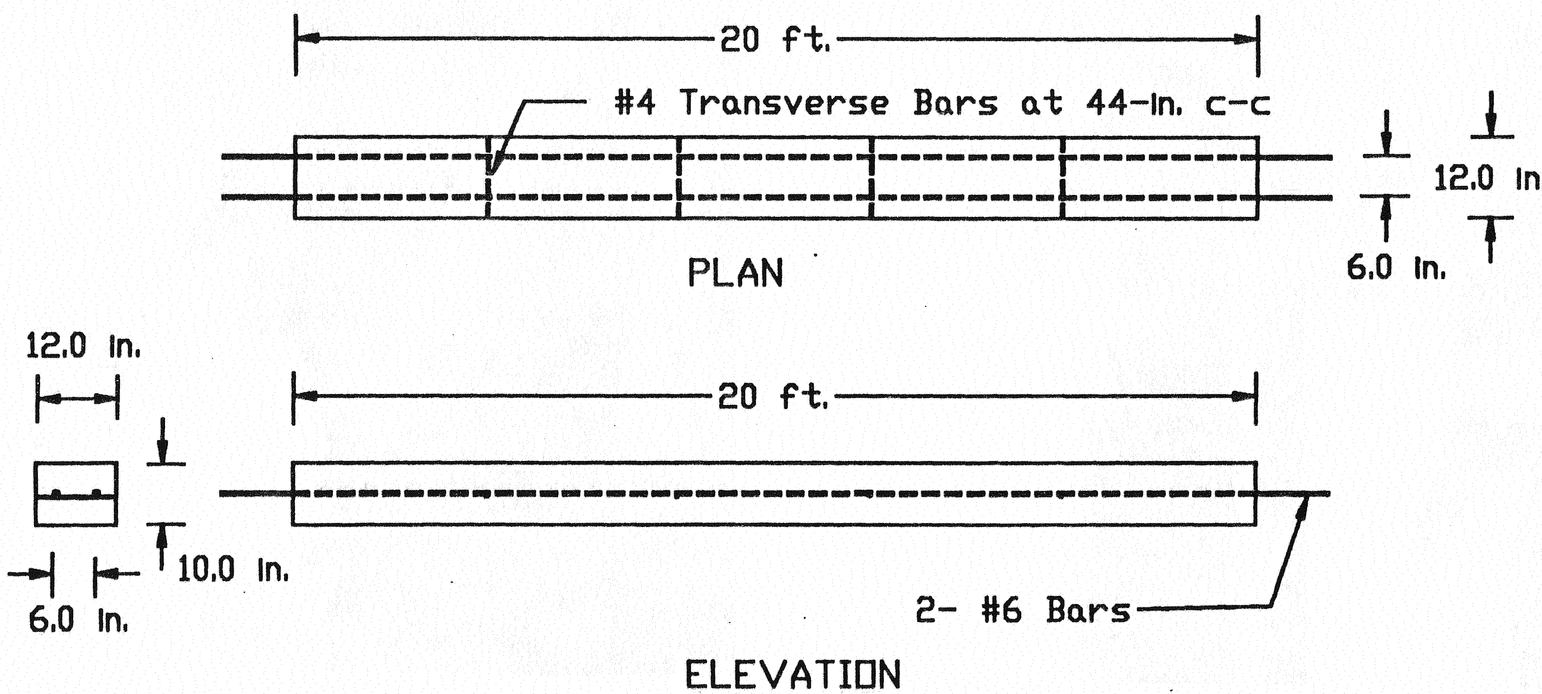
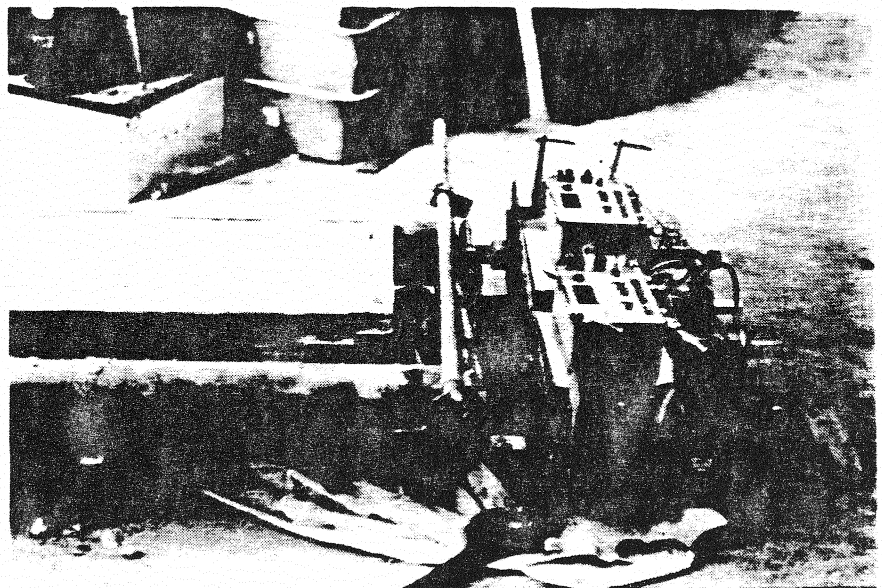
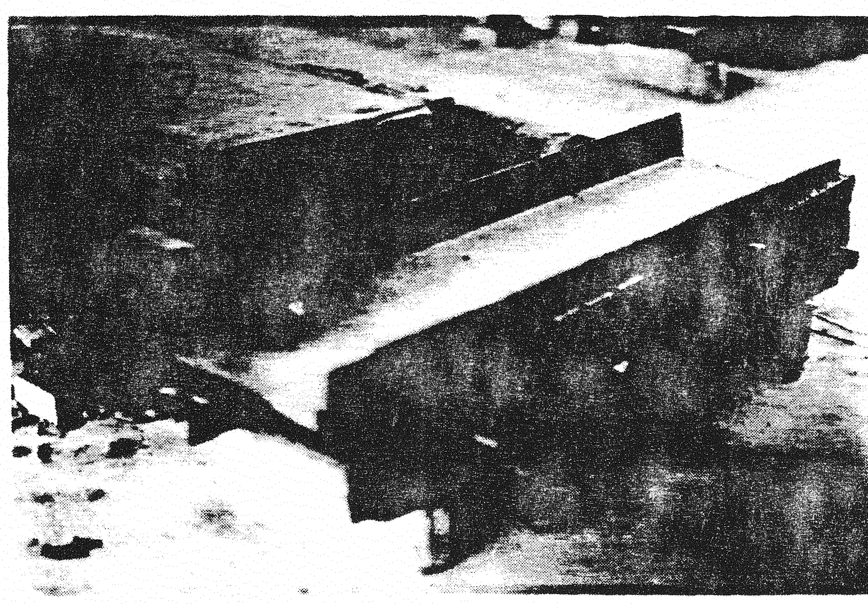


Figure 52. Slabs With 0.73% Reinforcement

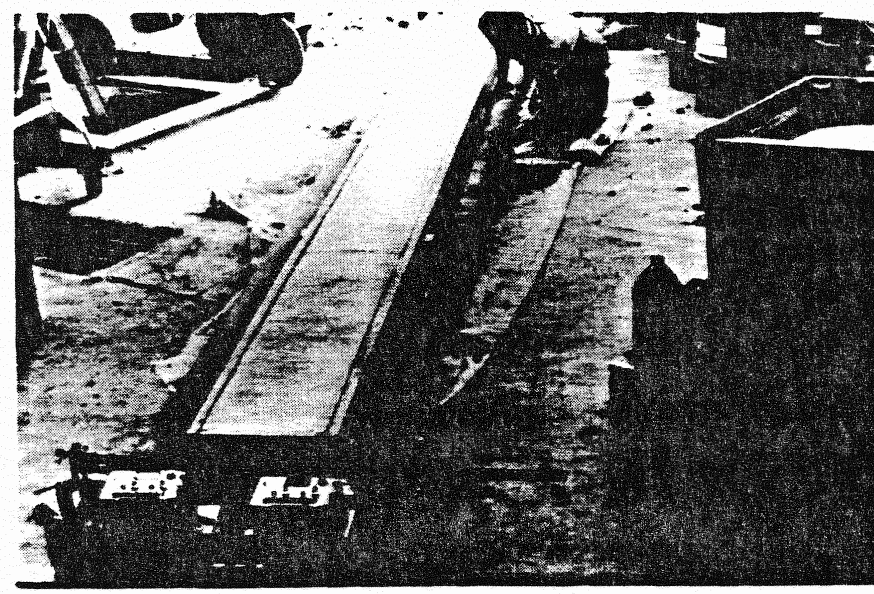


(a) Active End of Slab

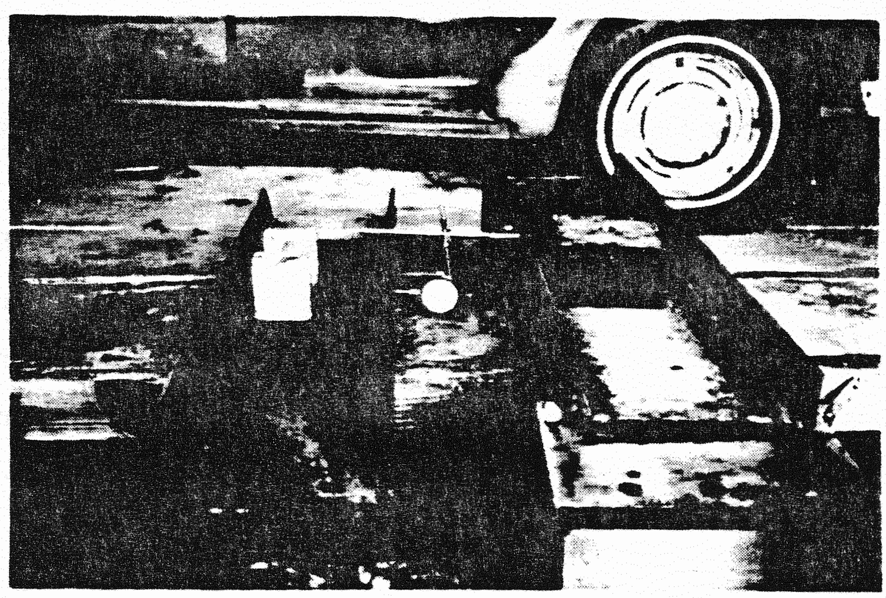


(b) Restrained End of Slab

Figure 53. Tensile Testing Apparatus



(c) Full View of Slab



(d) Dial Indicator Setup

Figure 53. Continued

ment control, the displacement at the time of cracking can be held while the crack position and width are recorded.

The load frames are self-contained units, independent of the floor system. On both ends of the frame, steel channels are placed above and below the reinforcing steel extending out of the specimen. The reinforcement is threaded on its ends and a nut is screwed onto the reinforcement. On one end of the specimen, the nuts are tightened against the outside flanges of the channels; on the other end of the specimen, hollow core load cells are placed around the bars and the nuts are used to hold the load cells tight against the outside flanges of the channels. Steel pipes are placed on both sides of the specimen, with one end of the pipe in contact with the inside flanges of the channels. On the other end of the pipes a threaded rod protrudes from a cap plate welded to the end of the pipe. The threaded rods extend between the channels in the same plane as the reinforcement. Nuts on the threaded rods bear against the inside flanges of the channels.

When the nuts bearing on the inside surfaces of the channels are turned counterclockwise, the nuts push on the channels, causing the channels to push on the load cells. The load cells are restrained from sliding by the nuts on the ends of the reinforcing bars. The net result is that the reinforcing bars are loaded in tension. Inside the slab, this tension is shared by the concrete. The magnitude of the tension is monitored with strain indicators wired to the load cells.

Displacement relative to the floor is monitored with two 1/100-in. dial gages, each placed at 2 ft from the ends of the 20-ft slab. Summing the movement at the gage locations allows determination of any length change between the gages. If a crack forms between the gage locations, the measured change in length is an indication of crack width. Crack widths were also measured from photographs taken through a comparator.

5.3 Test Procedure

All four slabs were placed at the same time with concrete supplied by a single ready-mix truck. An ODOT Class A mix was specified. Samples taken from the first and last portions of the load had the following properties:

First Portion of Load	Last Portion of Load
Temperature = 67°F	Temperature = 65°F
Slump = 4 in.	Slump = 3 in.
Air Content = 2.6%	Air Content = 3.4%
Unit Wt.= 135 pcf	Unit Wt. = 137 pcf

ODOT Specifications require Class A concrete to have an air content between 5 and 7% and slump between 1 and 3 in.

Concrete was vibrated during placement and manually leveled with a screed. The surface was troweled to achieve a smooth finish. The concrete was allowed to cure overnight under plastic sheets.

Specimens were tested within 18 to 24 hrs after placement. Loads were applied incrementally by turning the loading nut on one side of the specimen until 2 kips of load registered in the adjacent load cell. At this time the nut on the opposite side of the specimen was turned until 2 kips of load was induced in its adjacent load cell. When a combined total of 4 kips was present in the load cells, dial indicator readings were recorded. The procedure was repeated to a total load of 8, 12, 16 kips, etc., until the specimen cracked.

Immediately after cracking, dial indicator and load cell readings were recorded. In addition, the location of the crack relative to one end of the specimen was recorded. Unfortunately, dial indicator and load cell readings immediately before cracking could not be recorded since it was not known at what load the specimen would crack.

Loading and recording processes were repeated until at least two additional cracks were formed in each specimen. Specimens were left in the loaded state for several days after the primary testing, without the formation of additional cracks.

5.4 Results

Standard 6- by 12-in. cylinders were cast at the same time as the slabs. Results of compression and indirect tension tests on these cylinders are shown below:

3-day compressive strength	=	4462 psi
28-day compressive strength	=	4952 psi
1-day indirect tensile strength	=	131 psi
7-day indirect tensile strength	=	422 psi
14-day indirect tensile strength	=	442 psi
28-day indirect tensile strength	=	473 psi

Indirect tensile strengths were measured in split cylinder tests. ODOT Specifications require Class A concrete to have a 28-day compressive strength above 3000 psi. Specifications do not address limits for concrete tensile strength.

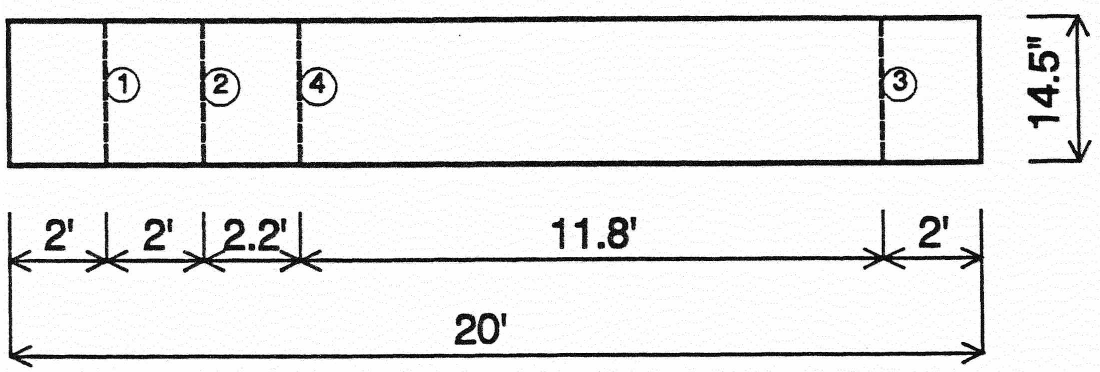
Drawings showing crack locations are provided in Figures 54 and 55. Dashed lines represent cracks and numbers near dashed lines identify the order in which the cracks formed. No significance should be attached to the fact that three cracks were formed in specimens constructed with coated steel and four cracks were formed in specimens constructed with uncoated steel. A fourth crack could have been formed in specimens constructed with coated steel by continuing to increase displacement after the third crack had formed.

Figures 56 through 59 are plots of tension applied to the specimens versus change in length between dial indicators. The plots show a linear relationship between force and displacement up to the formation of the first crack. The formation of a crack results in large displacements at the dial gages and corresponding reductions in force at the load cells. The numbers near the lines connecting data points indicate the order in which the cracks formed. The width of a crack is determined by subtracting the measured elongation immediately before the crack formed from the elongation immediately after the crack formed.

In Figure 57, there is obvious evidence of experimental error. Either a slip in the load frame or an error in applying displacements caused the load and elongation to decrease. Subsequent data are offset to lower total elongation values than would have been present if the error had not occurred. Displacements between individual data points which are not a part of the region of decreasing elongation should not be affected.

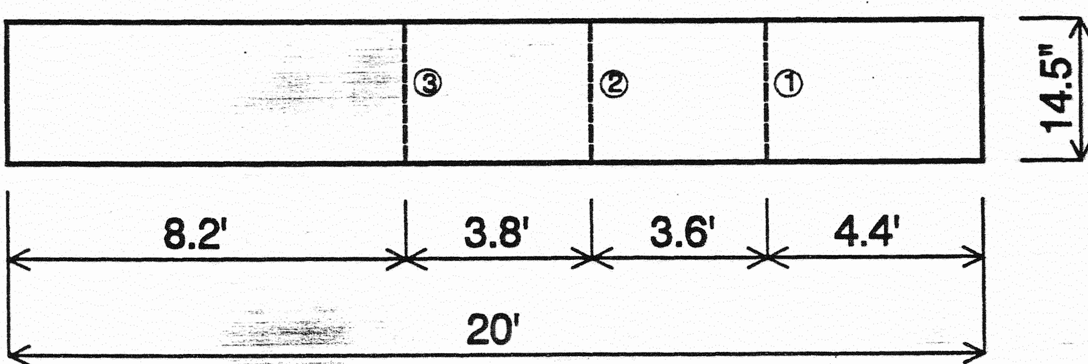
In Figure 60 through 63, examples of photographs taken through the comparator are presented. The numbers on both sides of the cracks identify the slab number and the crack position. Photographs were taken at arbitrary locations along the length of the cracks. Crack widths were determined from the known width of the grid in the comparator, seen above the cracks.

In Table 2, several of the key features of the plots in Figures 56 through 59 and the data from the comparator measurements are presented for comparison. In regard to crack width, the data do not indicate a significant difference between specimens constructed with uncoated



Plan View - Width of Slab Not to Scale

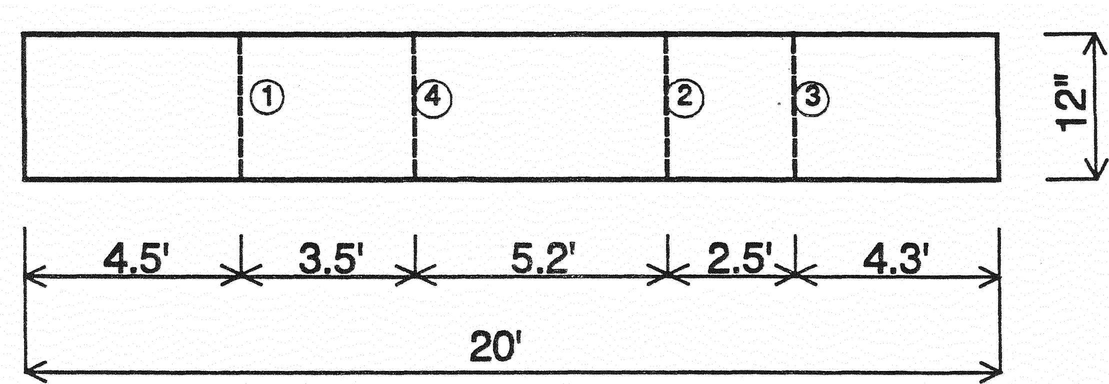
(a) Uncoated Steel



Plan View - Width of Slab Not to Scale

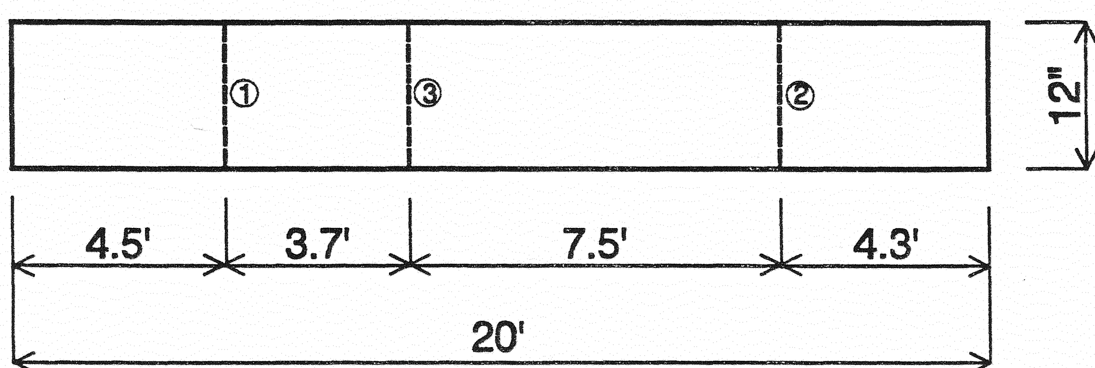
(b) Epoxy-Coated Steel

Figure 54. Crack Locations in Tensile Specimens, 0.61% Steel



Plan View - Width of Slab Not to Scale

(a) Uncoated Steel



Plan View - Width of Slab Not to Scale

(b) Epoxy-Coated Steel

Figure 55. Crack Locations in Tensile Specimens, 0.73% Steel

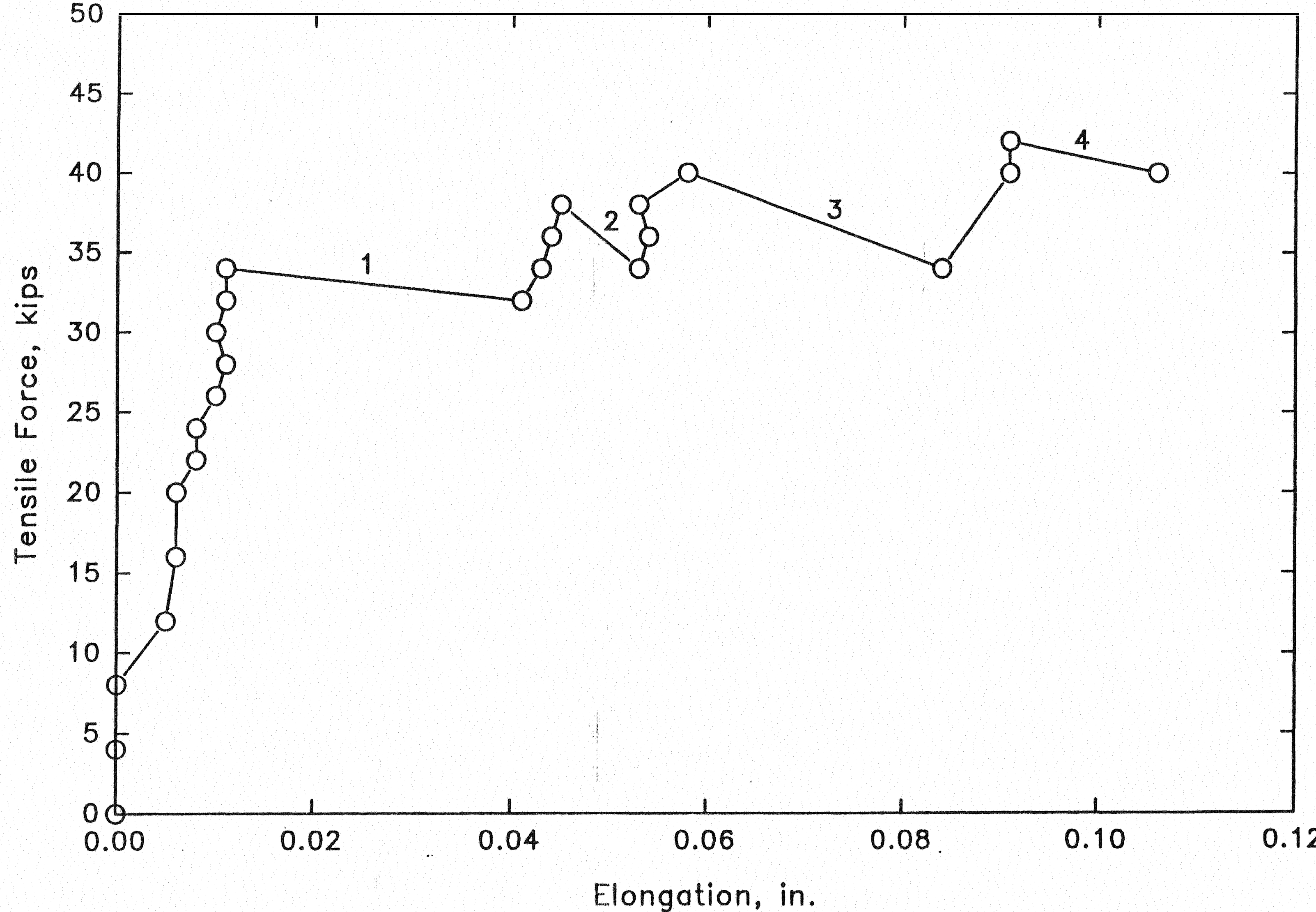


Figure 56. Tensile Test Results for Slab With 0.61% Uncoated Reinforcement

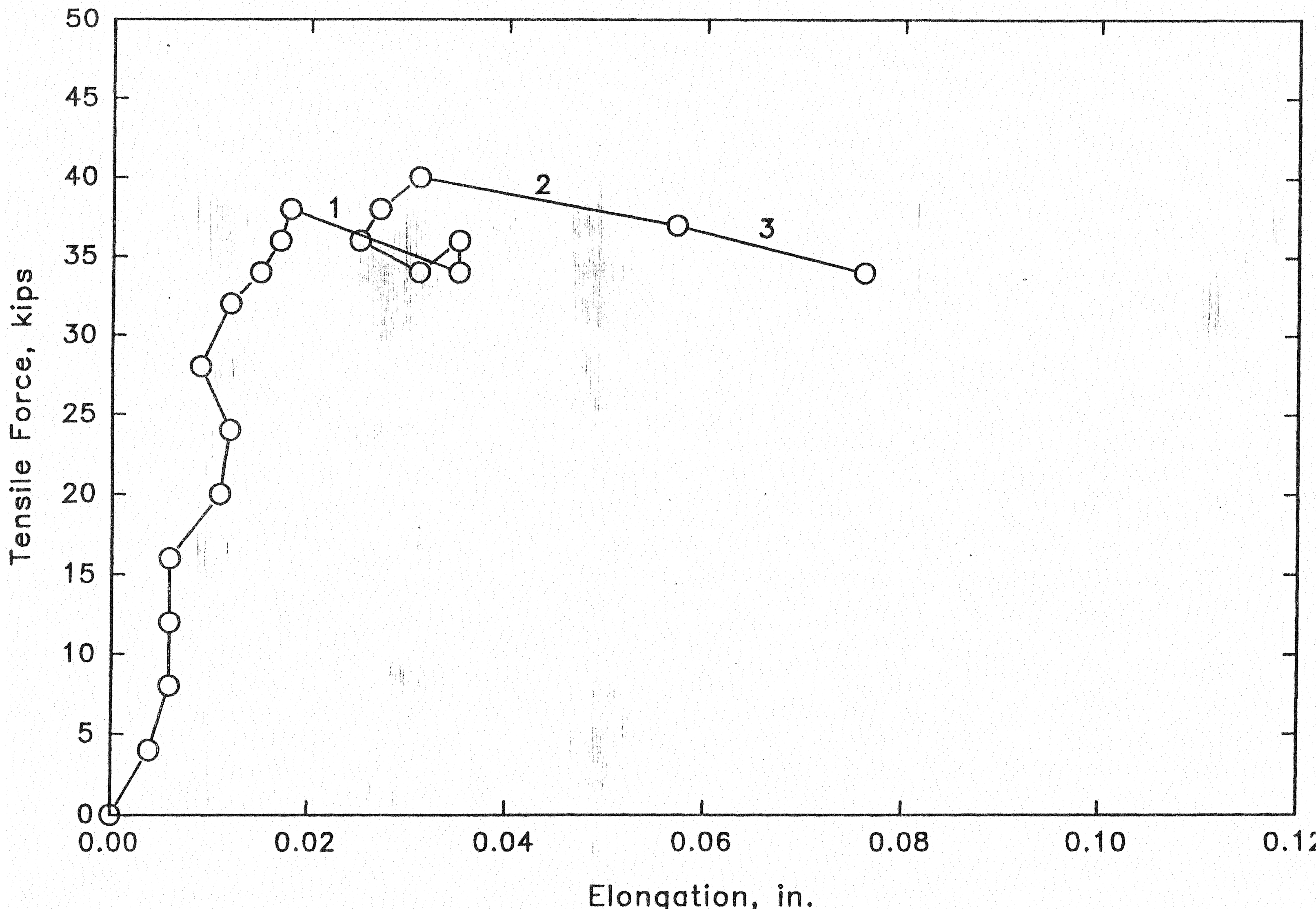


Figure 57. Tensile Test Results for Slab With 0.61% Coated Reinforcement

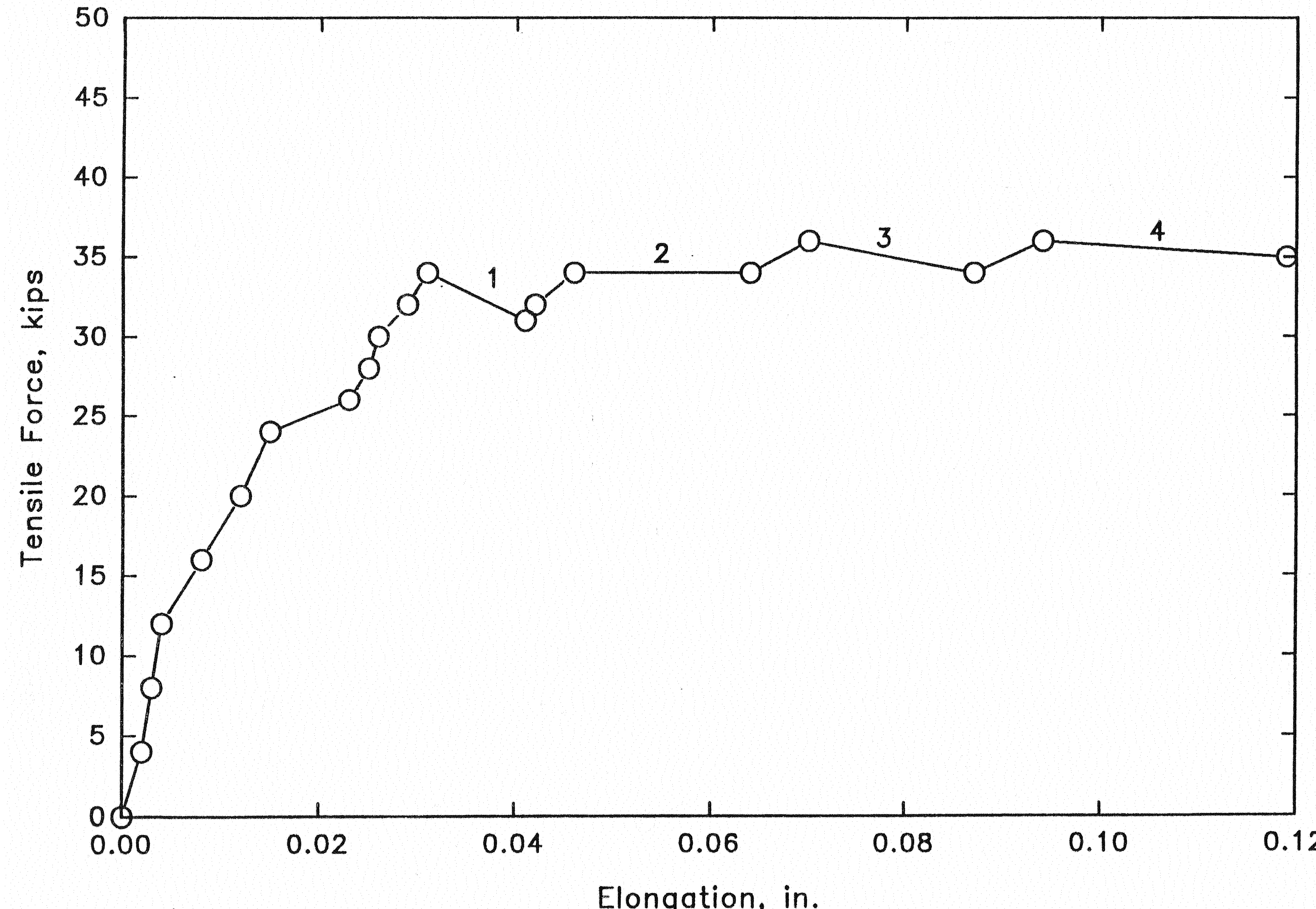


Figure 58. Tensile Test Results for Slab With 0.73% Uncoated Reinforcement

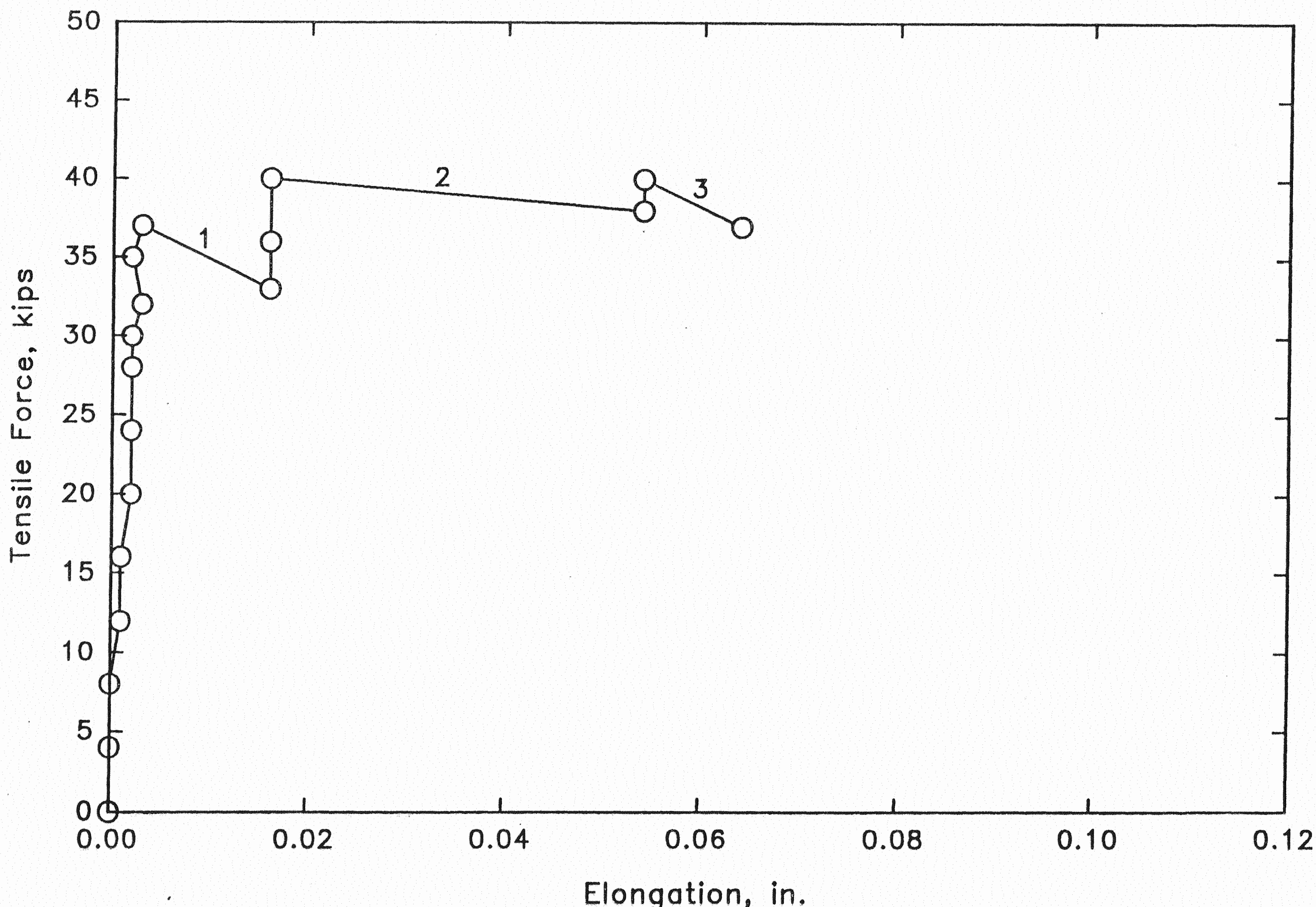
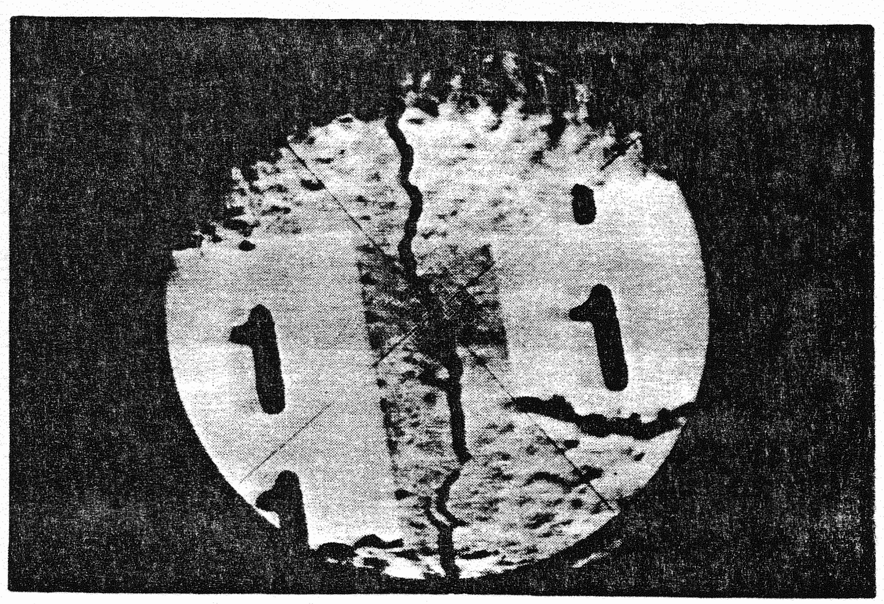
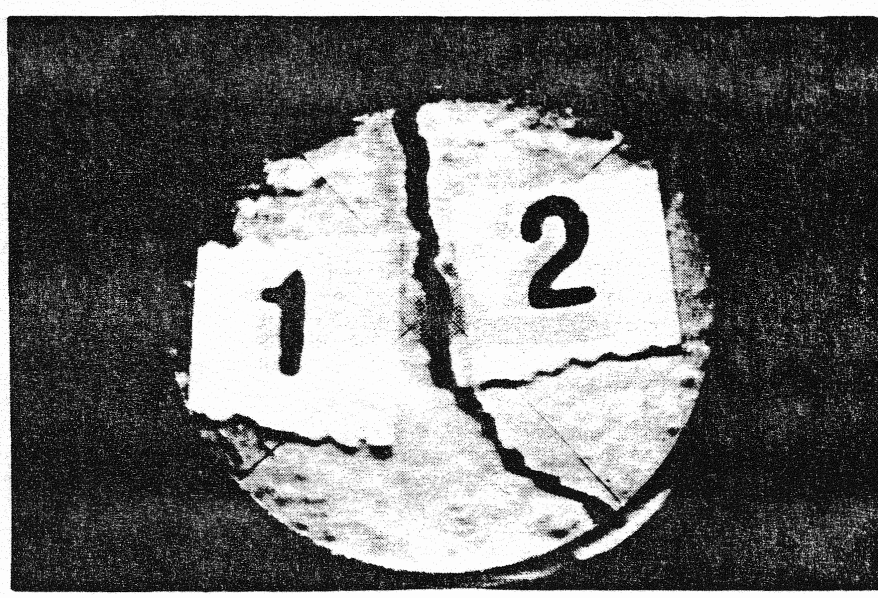


Figure 59. Tensile Test Results for Slab With 0.73% Coated Reinforcement



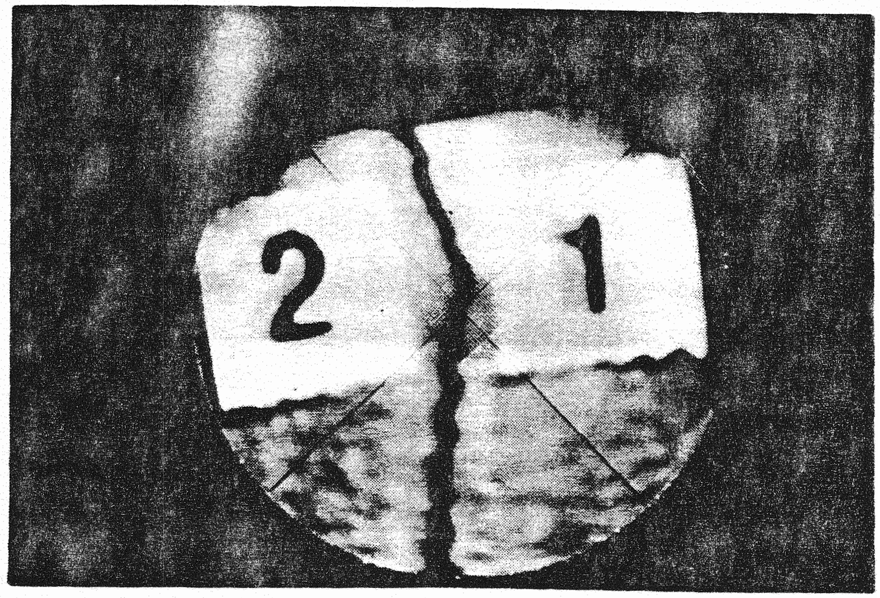
Crack Width = 0.27 mm



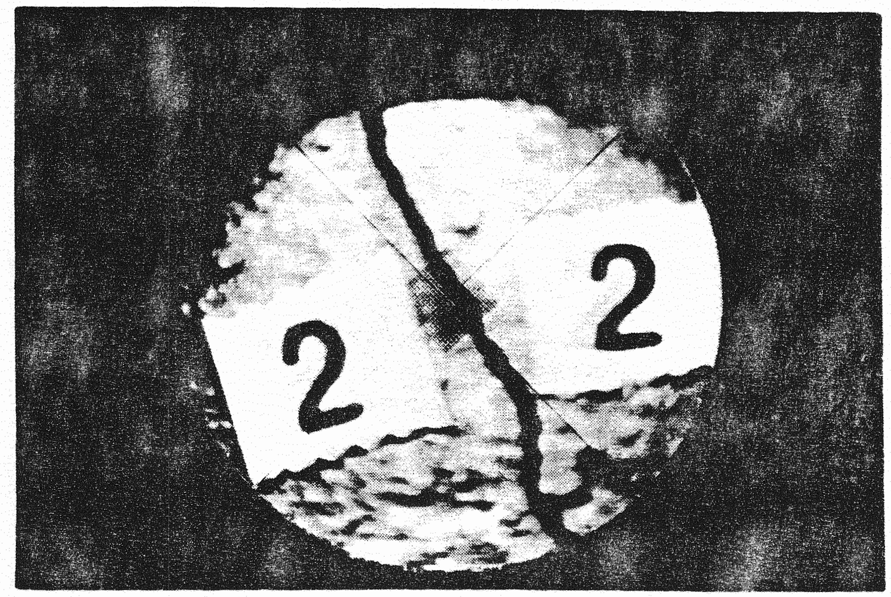
Crack Width = 0.39 mm

Average Crack Width = 0.36 mm

Figure 60. Typical Cracks Formed in Slab With 0.61% Uncoated Reinforcement



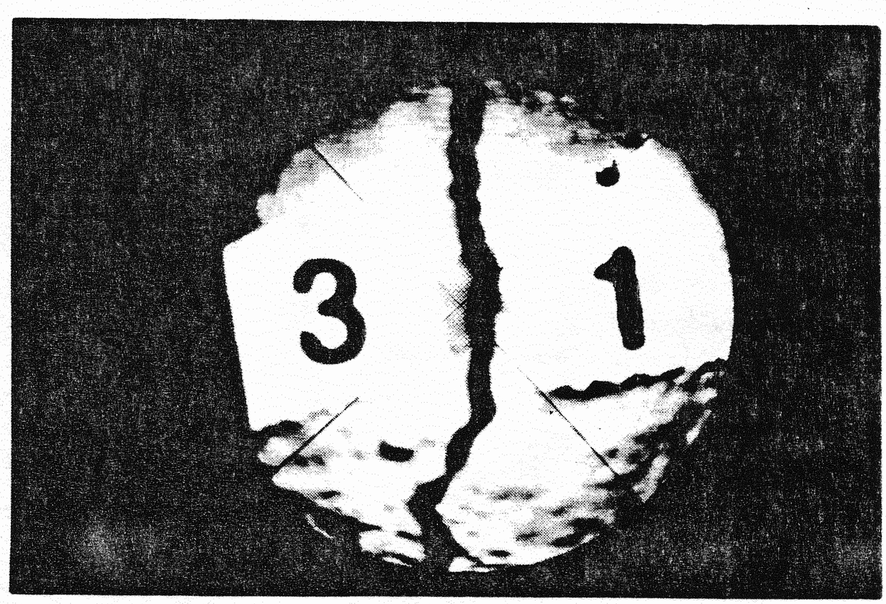
Crack Width = 0.27 mm



Crack Width = 0.32 mm

Average Crack Width = 0.24 mm

Figure 61. Typical Cracks Formed in Slab With 0.61% Coated Reinforcement

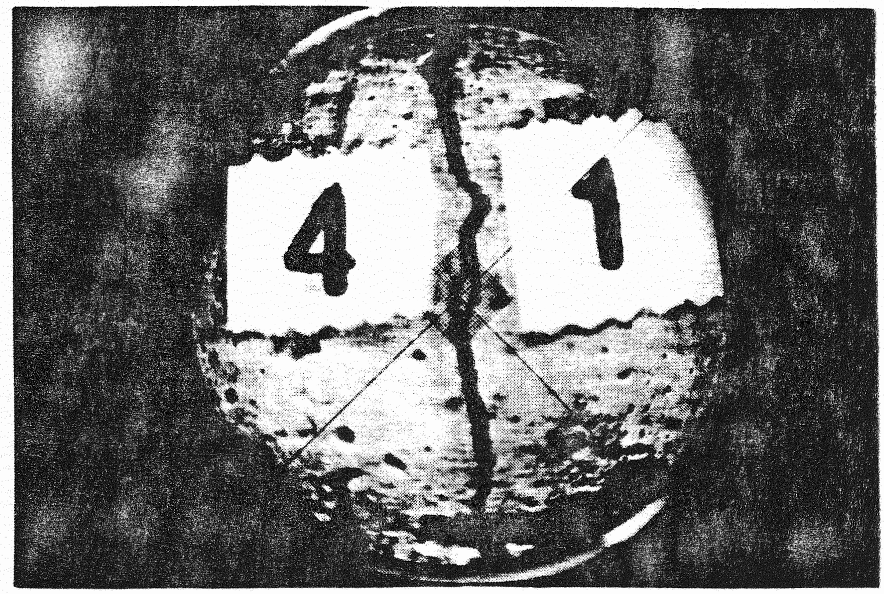


Crack Width = 0.53 mm

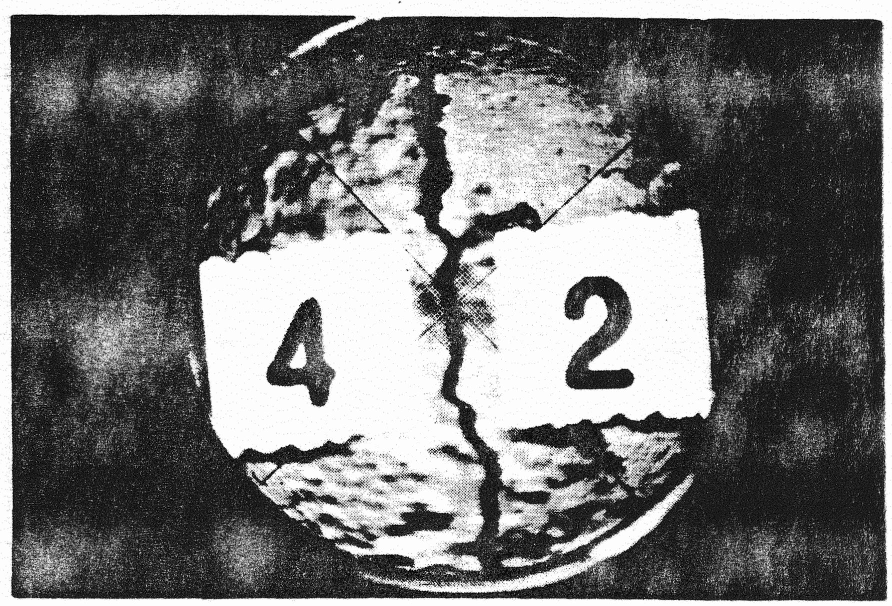
Crack Width = 0.30 mm

Average Crack Width = 0.36 mm

Figure 62. Typical Cracks Formed in Slab With 0.73% Uncoated Reinforcement



Crack Width = 0.31 mm



Crack Width = 0.27 mm

Average Crack Width = 0.26 mm

Figure 63. Typical Cracks Formed in Slab With 0.73% Coated Reinforcement

steel and those constructed with coated steel. There is a significant difference between measurements made with the comparator and those made with the dial indicators. This difference may be explained by the fact that the dial indicator readings represent an average width over the full length and depth of the crack, while the comparator measures crack width at only one location on the surface of the slab.

TABLE 2. RESULTS OF TESTS ON PAVEMENT SPECIMENS

Slab No.*		1	2	3	4
Average Crack Width, in.	Comparator	0.014	0.009	0.014	0.010
	Dial Indicator	0.020	0.021	0.018	0.020
Stiffness Prior to Cracking, K/in.		2490	2260	960	10800
Force to Produce First Crack, kips		34.0	38.0	34.0	37.0
Average Force to Produce Crack, kips		38.5	38.3	35.0	39.0

*Slab No. 1: 0.61% steel, uncoated.
Slab No. 2: 0.73 steel, coated.
Slab No. 3: 0.61% steel, uncoated.
Slab No. 4: 0.73% steel, coated.

In regard to stiffness prior to cracking, values for the first two slabs are not substantially different, indicating that epoxy coating is not affecting the stiffness of the slab. On the basis of the data for the first two slabs, a modular ratio of approximately 10 can be calculated, which is reasonable for the age of the concrete being tested. The third and fourth slabs have stiffnesses substantially different from the first two slabs, with the stiffness for the third slab being very low and that for the fourth slab being very high. The modular ratios for the third and fourth slabs are far outside the bounds of what should reasonably be expected; therefore, the stiffness data for these two slabs are discounted.

In regard to the force required to produce cracks, neither the force required to produce the first crack in a specimen nor the average force required to produce all the cracks in a specimen vary substantially between specimens. The force required to produce the first crack in specimens constructed with uncoated steel is consistently less than that required to produce the first crack in specimens constructed with coated steel; but with the limited data available and the small differences present, it cannot be stated with any certainty that the observed behavior represents a trend.

On the basis of the data presented here, it must be concluded that significant differences in performance do not exist among the four specimens tested. Neither epoxy coating nor the small change in concrete cross-sectional area had a significant effect on crack width or the force required to produce cracks. The effect of epoxy coating on stiffness prior to cracking appears to be negligible, but inconsistencies in these data limit the conclusions which can be drawn.

CHAPTER 6

DESIGN METHODS

In this chapter, the American Concrete Institute (ACI) [13] and Concrete Reinforcing Steel Institute (CRSI) [11] methods for design of CRCP are presented. Both of these methods are used to select pavement thickness and steel percentage, and the calculated values are compared to actual values used in the construction of the CRCP on Interstate 35. The CRSI method is further used to predict crack spacing and crack width for comparison to measured values.

The pavement under study is a 10-in. thick concrete slab on either an asphaltic concrete subbase (Project 1) or a lean concrete subbase (Project 2). The percentage of steel provided is 0.51 in northbound lanes of Project 1, and 0.61 in southbound lanes of Project 1 and both lanes of Project 2. The design methods rely very heavily on properties such as the load transfer coefficient for the cracked slab, the modulus of subgrade reaction, and the tensile strength of the concrete. None of these properties are readily available and all of them are subject to a great deal of variability. As a consequence, the following calculations are based on many assumed values.

6.1 ACI Committee 325, Subcommittee 7 Method

ACI Committee 325 has published a design procedure for CRCP [13]. This method was developed from a number of other design methods and from information gathered on the actual performance of pavements. This method is based on the assumption that stress and deflection produced by wheel loads are resisted by the concrete slab. The function of longitudinal steel is to control crack spacing for effective load transfer at cracks.

6.1.1 Thickness Design

The ACI method for thickness design was developed from the results of the AASHTO road test. Modifications were made to the original equations to generate design equations specifically applicable to CRCP. The resulting thickness design equation is shown below. Required thickness can be calculated using this equation or the nomograph in Ref. [13]:

$$\log \Sigma L = -8.682 - 3.513 \log \left[\frac{J}{M_R h^2} \left(1 - \frac{2.61a}{Z^{0.25} h^{0.75}} \right) \right] - \frac{0.1612}{\beta'} \quad (1)$$

where

ΣL = number of accumulated equivalent 18-kip single axle loads;

J = a coefficient dependent on load transfer characteristics or slab continuity;

M_R = third point loading modulus of rupture of concrete at 28 days, psi;

h = nominal thickness of concrete pavement, in.;

Z = E_c/k;

a = radius of equivalent loaded area;

E_c = modulus of elasticity of concrete, psi;

k = modulus of subgrade reaction, psi/in.; and

β' = $1 + (1.624 \times 10^7/(h + 1)^{8.46})$.

Numerical values for insertion in Eq. (1) are provided in Table 3.

TABLE 3. THICKNESS DESIGN DATA

ΣL	=	49, 234, 387 [Estimated from traffic data provided by ODOT]
J	=	2.2 [Ref. 13]
M_R	=	650 psi [Ref. 13]
a	=	7.15 [Ref. 13]
E_c	=	3,950,000 psi [Ref. 11]
k	=	240 [Ref. 11]

Solving Eq. (1) by trial and error for h results in $h = 11.2$ in. This compares to a constructed thickness of 10 in.

6.1.2 Reinforcing Steel Design

The purpose of longitudinal steel in CRCP is to hold transverse cracks tightly closed so that load can be transferred across the cracks by shear interlock. To perform this function, sufficient steel must be available to resist stresses due to shrinkage and temperature changes.

Percentage of steel should be calculated from each of the equations shown below and the largest value selected for construction.

1. To prevent yielding of the steel at cracks due to restrained volume changes:

$$p_s = \left(\frac{f_t'}{f_y}\right)(100) \tag{2}$$

where

f_t' = tensile strength of concrete, psi;

f_y = yield strength of steel, psi; and

p_s = percentage of steel.

Using $f_t' = 0.4\ M_R = 260$ psi and $f_y = 60{,}000$ psi results in $p_s = 0.433\%$.

2. To resist shrinkage:

$$p_s = \left(\frac{f_t'}{f_y + \delta E_s - n f_t}\right)100 \tag{3}$$

where

δ = coefficient of shrinkage of concrete;

E_s = Young's modulus of elasticity for steel; and

n = E_s/E_c.

Using values of f_t' and f_y as shown above, along with $\delta = 0.0003$ [11] and $E_s = 29{,}000{,}000$ psi, results in $p_s = 0.389\%$.

3. To control restrained volume changes due to temperature:

$$p_s = \left(\frac{f_t'}{f_y - n f_t'}\right)100 \tag{4}$$

and

$$p_s = \left[\frac{f_t'}{2\,(f_y - \Delta T \alpha_s E_s)}\right]100 \tag{5}$$

where ΔT is the total temperature drop in degrees Fahrenheit from temperature at time of construction, and α_s is the thermal coefficient of steel. Using the same values for f_t', f_y, and E_s as above, and taking ΔT = 100°F and α_s = 0.0000065 in./in./°F, results in p_s = 0.448% from Eq. (4) and p_s = 0.316% from Eq. (5).

The largest value for p_s is 0.448 from Eq. (4). Reference [13] suggests that a factor of safety of 1.3 be applied to the calculated steel percentage. Applying this factor of safety, the required steel percentage is (1.3) (0.448) = 0.582. The actual steel percentage provided is 0.51 for Project 1 northbound lanes, and 0.61 for Project 1 southbound lanes and all lanes of Project 2.

6.2 CRSI Method

This design procedure can be used to calculate the pavement thickness and percentage of steel required in CRCP. The method was developed under the direction of the Associated Reinforcing Bar Producers and first introduced by McCullough and Cawley [11]. This procedure provides guidance for the selection of subbase, slab thickness, percentage of steel, construction joints, and terminal treatment. Only slab thickness and percentage of steel will be considered here.

6.2.1 Thickness Design

Thickness can be calculated using the equation shown below or the nomograph in Ref. [11]. A terminal serviceability index of 2.5 is assumed in the calculation:

$$\log \Sigma L = 7.35 \log (h+1) - 0.06 - \frac{0.0176}{\beta'}$$

$$+ 3.42 \log\left[\frac{f_w}{215.6\, J'}\left(\frac{h^{0.75} - 1.132}{h^{0.75} - 18.42 / Z_d^{0.25}}\right)\right] \qquad (6)$$

where

f_w = working stress in concrete, psi;

J' = load transfer coefficient;

Z_d = E_c/k_d; and

k_d = design k-value including a consideration of erodability, psi/in.

In Ref. [11], it is recommended that f_w be taken as 0.75 M_R, J' is conservatively taken as 3.2 for edge loading, and k_d = k for a cement-treated or asphalt-stabilized base.

Using the same values for ΣL, M_R, E_c, and k as in Section 6.1, and iteratively solving for h, the required slab thickness is determined to be 11.85 in. This compares to a constructed thickness of 10 in.

6.2.2 Reinforcing Steel Design

The object of the steel design procedure is to establish a percentage and distribution of steel which will result in acceptable levels of crack spacing, crack width, and steel stress. Steel percentage can be calculated using the equations shown below or the nomographs presented in Ref. [11].

1. Design based on crack spacing. Limits on crack spacing are established to prevent spalling and punchouts. In Ref. [11], a maximum crack spacing of 8 ft and a minimum crack spacing of 3.5 ft are recommended. The maximum crack spacing will correspond to a minimum steel percentage, and the minimum crack spacing will correspond to a maximum steel percentage:

$$\overline{X} = \frac{1.32\left(1+\frac{M_R}{1000}\right)^{6.70}\left(1+\frac{\alpha_s}{2\alpha_c}\right)^{1.15}(1+\phi)^{2.19}}{\left(1+\frac{\sigma_w}{1000}\right)^{5.20}(1+p_s)^{4.60}(1+1000\,\delta)^{1.79}} \quad (7)$$

where

$\overline{X}$ = crack spacing, ft;

α_s/α_c = ratio of thermal coefficients of steel to concrete;

ϕ = reinforcing bar diameter, in.;

σ_w = wheel load stress, psi; and

p_s = percent steel.

Based on $\alpha_s/\alpha_c = 1.0$, F = 0.75 in., and σ_w = 190 psi (for a 10-in. slab thickness, 18,000-lb wheel load, and k = 24) [11], p_s = 0.50% when $\bar{X}$ = 8.0 ft and p_s = 0.80% when $\bar{X}$ = 3.5 ft.

2. Design based on crack width. An upper limit on crack width is established to prevent spalling. In Ref. [11], the maximum allowable crack width is related to the design temperature drop. For a design temperature drop of 100°F, the allowable crack width is 0.055 in.:

$$CW = \frac{0.00932\left(1+\frac{M_R}{1000}\right)^{6.53}(1+\phi)^{2.20}}{\left(1+\frac{\sigma_w}{1000}\right)^{4.91}(1+p_s)^{4.55}} \quad (8)$$

where CW is crack width. Solving with CW = 0.055 in. results in p_s = 0.51%.

3. Design based on steel stress. An upper limit on steel stress is established to prevent steel yielding and excessive permanent deformation. In Ref. [11], the allowable stress is related to the indirect tensile strength of the concrete. Assuming a modulus of rupture of 650 psi, the allowable steel stress is 58.5 ksi:

$$\sigma_s = \frac{47{,}300\left(1+\frac{\Delta T_D}{100}\right)^{0.425}\left(1+\frac{M_R}{1000}\right)^{4.09}}{\left(1+\frac{\sigma_w}{1000}\right)^{3.14}(1+1000\delta)^{0.494}(1+p_s)^{2.74}} \quad (9)$$

where σ_s is allowable steel stress, ksi; and ΔT_D is design temperature drop, °F. Based on σ_s = 58.5 ksi and ΔT_D = 100°F, p_s = 0.70%.

The value for p_s calculated on the basis of minimum crack spacing is the maximum steel percentage permitted; all other calculated values of p_s are minimums. Minimum steel percentage is controlled by allowable stress in the steel and is equal to 0.70%; maximum steel percentage is 0.80%. The steel percentage provided is permitted to fall anywhere between this maximum and minimum. The actual steel percentage provided is 0.51 for Project 1 northbound lanes, and 0.61 for Project 1 southbound lanes and all lanes of Project 2.

6.3 Estimation of Crack Width and Crack Spacing

The CRSI method may also be used to calculate an expected crack spacing and width based on a given steel percentage.. For a steel percentage of 0.51, as on the northbound lanes of Project 1, predicted crack spacing is 7.8 ft and crack width is 0.055 in. For a steel percentage of 0.61, as on the southbound lanes of Project 1 and all of Project 2, predicted crack spacing is 5.8 ft and crack width is 0.04 in.

The average crack spacings obtained from the crack survey are 5.0 and 5.7 ft for south- and northbound lanes of Project 1, and 6.1 and 6.9 ft for south- and northbound lanes of Project 2. Sampled crack widths on the pavements vary from 0.008 to 0.012 in. Caution should be used in comparing values obtained from the crack survey to values estimated using the standard design method, since approximately two years are required for crack growth to stabilize. Project 1 was just over two years old and Project 2 was just over one year old at the time of the measurements.

CHAPTER 7

SUMMARY, CONCLUSIONS, AND FUTURE WORK

7.1 Summary

The results of this investigation may be summarized as follows:

1. A literature search was conducted for materials related to CRCP and epoxy-coated reinforcement. This search revealed that CRCP has been and is being used extensively without major performance problems. Problems which have occurred have generally been attributable to construction errors. In areas where heavy use of deicing salts accelerates corrosion of reinforcement, epoxy coating has proven to be an effective means of protecting the steel.

2. Crack surveys were conducted over the full length of two recently constructed CRCPs. Both projects are approximately six miles long. Northbound lanes are reinforced with epoxy-coated steel and southbound lanes are reinforced with uncoated steel. The older of the two projects had been completed approximately two years and the younger approximately one year before the latest survey. Crack spacing appears to be stabilizing on the older pavement, with a projected average spacing between 4 and 6 ft. The average crack spacing on pavements constructed with epoxy-coated steel is less than on pavements constructed with uncoated steel, but the difference is small. The data do not provide evidence of definite relationships between crack spacing and concrete strength, air temperature, air content, or slump.

3. Instrumentation was installed for monitoring crack widths and corrosion potential. Crack widths are monitored by measuring the distance between gauge points installed on opposite sides of arbitrarily selected cracks. Initial crack widths, as measured through a comparator, ranged between 0.008 and 0.012 in. To monitor corrosion potential, one half-cell test site was installed on both the north- and southbound lanes of one project. Initial measurements indicated little or no corrosion activity. The locations of half-cell and gauge sites are provided in Chapter 4 and the Appendix.

4. Tests were conducted on four laboratory specimens to permit pavement constructed with epoxy-coated steel to be compared to pavement constructed with uncoated steel in a more controlled environment. The two sets of specimens exhibited no significant differences in average crack width, stiffness prior to cracking, or force required to cause cracking.

5. ACI and CRSI methods were used to design CRCPs for comparison to the constructed CRCPs. According to both methods, the constructed pavement thickness is less than required. According to the ACI method, the percentage of steel is slightly more than required, while the CRSI method indicates the steel percentage is low. The CRSI method may also be used to calculate an expected crack spacing and width for a given design. Calculated spacing and width are both slightly greater than measured values.

7.2 Conclusions

Based on the results of the literature search, field surveys, and laboratory tests, it is concluded that in terms of crack spacing and width, pavement constructed with epoxy-coated steel may be expected to perform in a manner similar to pavement constructed with uncoated steel. If, in the future, the reinforcement should begin to corrode, the epoxy-coated steel would have the advantage.

7.3 Future Work

It is recommended that ODOT continue monitoring the performance of the pavement. A crack survey should be conducted within the next two years to determine the stabilized crack spacing. Crack widths should be sampled at arbitrary locations to determine if the width will remain below calculated values. Half-cell sites should be monitored for signs of corrosion activity.

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APPENDIX

GAUGE POINT STATIONS AND INITIAL GAUGE READINGS

Project	Station*	Initial Gauge Readings	
		On Gauge	On Pavement
Project 2—Northbound			
Crack 1	990+11.4	0.0536	0.1119
Crack 2	975+133.3	0.0536	0.1265
Crack 3	945+48.7	0.0536	0.1342
Crack 4	890+456.3	0.0536	—
Crack 5	710+334.2	0.0536	0.1648
Crack 6	710+331.4	0.0536	0.1467
Project 2—Southbound			
Crack 1	970+60	0.1198	0.2183
Crack 2	935+274	0.1007	0.2013
Crack 3	905+272	0.1087	0.1364
Crack 4	870+462	0.1122	0.1781
Crack 5	840+429	0.1089	0.2230
Crack 6	800+339	0.1161	0.1956
Crack 7	795+240	0.1074	0.1199
Crack 8	750+59	0.1004	0.2355
Crack 9.	705+200	0.1044	0.2177

*Station numbers are stamped in pavement at 500-ft intervals.

When using a Whittemore gauge, a measurement is first taken on a standard bar; these measurements are listed in the "On Gauge" column shown above. Values listed in the "On Pavement" column were measured between gauge points marked on pavement inserts. Changes in crack width relative to initial readings are calculated as:

D(CW) = (Current "On Pavement" Measurement - Current "On Gauge" Measurement)
- (Initial "On Pavement" Measurement - Initial "On Gauge" Measurement)

A positive result indicates an increase in width; a negative result indicates a decrease in width.