PMFSEL REPORT NO. 87-1
JANUARY 1987

BOND STRENGTH OF EPOXY-COATED REINFORCING BARS

By

Robert A. Treece
James O. Jirsa

Report on a Research Project
Sponsored by
Concrete Reinforcing Steel Institute
Reinforced Concrete Research Council- Project 50

PHIL M. FERGUSON STRUCTURAL ENGINEERING LABORATORY
Department of Civil Engineering / Bureau of Engineering Research
The University of Texas at Austin

A B S T R A C T

In this study, 21 beam specimens were tested to determine the bond strength of epoxy-coated and uncoated reinforcing bars in tension. Specimens were constructed with either #6 or #11 bars spliced in the center of the beam. Bars were uncoated with normal mill scale or epoxy-coated with a nominal coating thickness of 5 or 12 mils. The concrete strength ranged from 4000 psi to 12,600 psi. Seventeen specimens were top cast (more than 12 in. of concrete below) and four were bottom cast.

Performance was evaluated using measured bond strength, crack width and spacing, and stiffness of the beams. Results indicated that epoxy-coated bars developed approximately 65% of the bond of uncoated bars where failure was governed by splitting of the concrete cover. The bond reduction was independent of concrete strength, bar size, and coating thickness. The average width of cracks in coated bar specimens was 50% greater than in uncoated bar specimens; however, comparison of load-deflection diagrams showed no loss of stiffness when using epoxy-coated bars.

ACKNOWLEDGEMENTS

The research was conducted as part of the requirements for the M.S. degree under the direction of Dr. James O. Jirsa. Appreciation is expressed to Dr. John E. Breen for his interest in this project and his editorial comments, and to Dr. Michael E. Kreger for his help.

The research study was sponsored by the Concrete Reinforcing Steel Institute and the Reinforced Concrete Research Council under Project 50. The advice and comments of Task Group Chairman John F. McDermott and Members David P Gustafson, Roger H. Wildt, Henry G. Russell, Robert J. Smith, and Paul Zia were sincerely appreciated. Mr. B.K. Thornley of Florida Steel Corporation was most helpful in providing the reinforcing bars used in the study. The careful attention of Mr. V.A. Walter, Jr., of CRSI, to details associated with the support of the project helped to make the project proceed rapidly. The work was conducted in the Ferguson Structural Engineering Laboratory. Special thanks go to FSEL Staff members Gorham Hinkley, Blake Stasney, Dick Marshall, Pat Ball, Laurie Golding, Maxine DeButts, Sharon Cunningham, Jean Gehrke, and Peggy M. Carrasquillo. Although many graduate students helped during casting and testing, special appreciation is extended to David L. Hartmann and Dan R. Stoppenhagen who gave unselfishly of their time to assist with casting specimens and testing.

L I S T   O F   T A B L E S

Table		Page
2.1	Test Parameters	14
2.2	Reinforcing Bar Properties	21
2.3	Thickness of Epoxy Coating	21
3.1	Actual Specimen Parameters and Measured Test Data	41
3.2	Comparison of Results	44
3.3	Previous Tests on HIgh-Strength Concrete	48
3.4	Average Crack Widths	67
4.1	Comparisons Using Adjusted Bond Values	76

LIST OF FIGURES

Figure		Page
1.1	Inclination of Bond Stresses	3
1.2	Splitting Failure	3
1.3	Bond Stresses in Pullout Failure	3
1.4	Splitting Failure Patterns	5
1.5	Pullout Specimen (Ref. 5)	7
1.6	Beam End Specimen (Ref. 6)	7
2.1	Test Setup	13
2.2	Beam Cross Sections	17
2.3	Beam Dimensions	19
2.4	Mikrotest Thickness Gage	22
2.5	Measuring Coating Thickness	22
2.6	Distribution of Coating Thickness Measurements, Specimen 12-6-4	23
2.7	Distribution of Coating Thickness Measurements, Specimen 5-6-4	27
2.8	Distribution of Coating Thickness Measurements, Specimen 12-6-4r	27
2.9	Distribution of Coating Thickness Measurements, Specimen 5-6-4r	28
2.10	Distribution of Coating Thickness Measurements, Specimen 12-11-4	30
2.11	Distribution of Coating Thickness Measurements, Specimen 5-11-4	31

LIST OF FIGURES (continued)

Figure		Page
3.19	Beam End Deflection, Series X-11-12	59
3.20	Beam End Deflection, Series X-11-12b	60
3.21	Average Crack Widths, Series X-6-4	62
3.22	Average Crack Widths, Series X-6-4r	62
3.23	Average Crack Widths, Series X-11-4	63
3.24	Average Crack Widths, Series X-11-4b	63
3.25	Average Crack Widths, Series X-6-8	64
3.26	Average Crack Widths, Series X-11-8	64
3.27	Average Crack Widths, Series X-6-12	65
3.28	Average Crack Widths, Series X-11-12	65
3.29	Average Crack Widths, Series X-11-12b	66
3.30	Crack Width vs. Coating Thickness	69
3.31	Components of Bond With Friction	71
3.32	Components of Bond Without Friction	71
3.33	Tensile Stresses Across Splitting Plane	73

CHAPTER 1

INTRODUCTION

1.1 Usage of Epoxy-Coated Reinforcing Bars

The primary purpose of epoxy-coated bars is to prevent corrosion of the steel which leads to premature deterioration of concrete structures. When steel corrodes, the corroded material expands up to twenty times the original volume of the steel. This expansion exerts a radial pressure on the concrete, which causes cracking and spalling. Chloride ions, carried by water, reach the reinforcing steel through cracks in the concrete and produce corrosion. Sources of chloride ions include de-icing salts used on highways, bridge decks, parking garage slabs, and seawater spray in coastal regions.

Epoxy-coated bars have been primarily used in bridge decks to prevent corrosion due to de-icing salts. They were first introduced in 1973, in a bridge deck in Pennsylvania. Since then, they have been used in nearly all types of structures. In coastal regions all elements of a bridge exposed to sea water or sea spray may be built with epoxy-coated bars to prevent corrosion. Epoxy-coated bars have also been used in structures where concrete is exposed to a corrosive environment. Applications include sewage treatment plants, water-chilling stations, and chemical plants.

1.2 Review of Bond

The bond of reinforcing bars to concrete is critical in the analysis and design of reinforced concrete structures. Inherent in the analysis of a reinforced concrete section is the assumption that the strain in the concrete and steel is equal at the location of the steel. This implies perfect bond between the concrete and steel.

1.2.1 ACI Code Provisions To insure ductility, bond between the steel and concrete must be maintained until the bars develop yield. ACI Building Code Requirements for Reinforced Concrete (ACI 318-83)[1] insures this ductility by specifying a required development length or splice length for all bars. The development length required is based on the bond strength the bars are capable of developing. Bond strength is dependent on bar size, depth of cover, spacing between bars, transverse

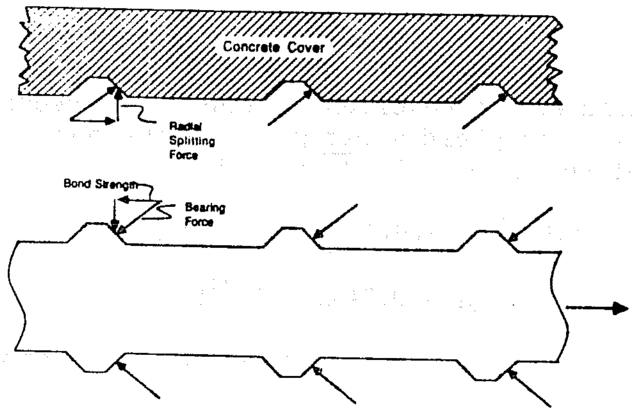


Figure 1.1 Inclination of Bond Stresses

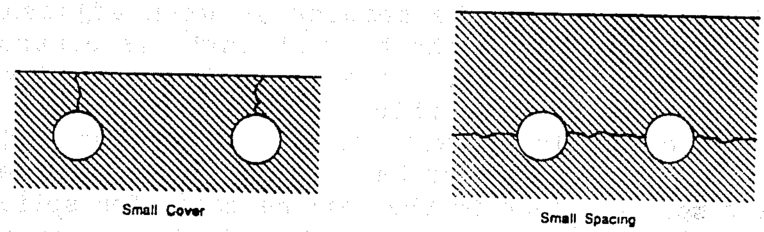


Figure 1.2 Splitting Failure

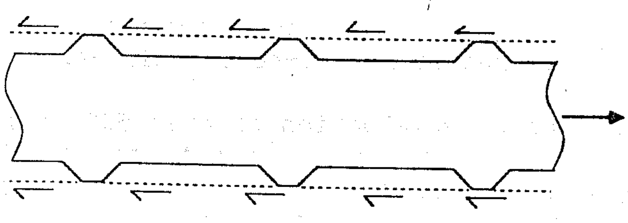


Figure 1.3 Bond Stresses in Pullout Failure

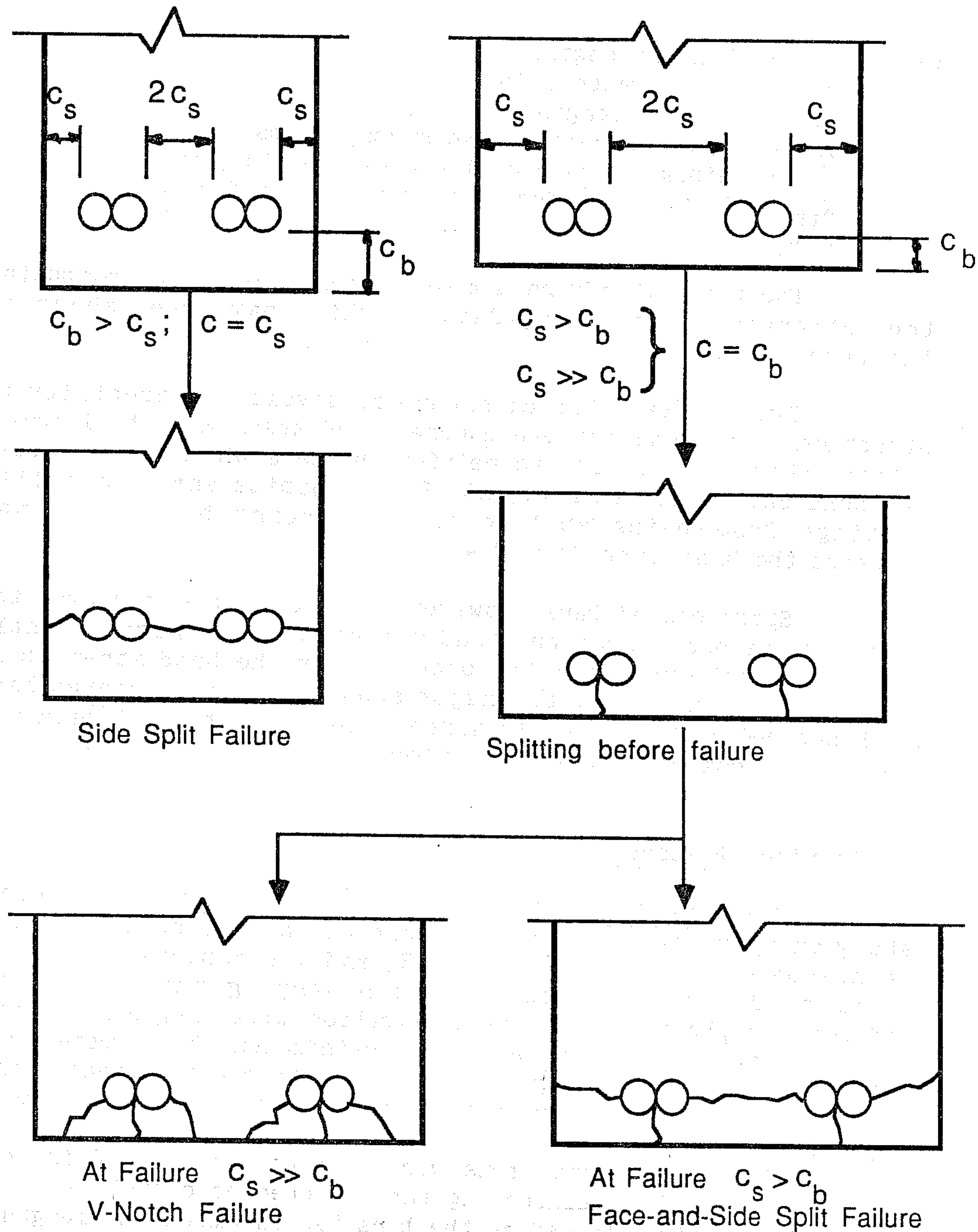


Fig 1.4 Splitting Failure Patterns

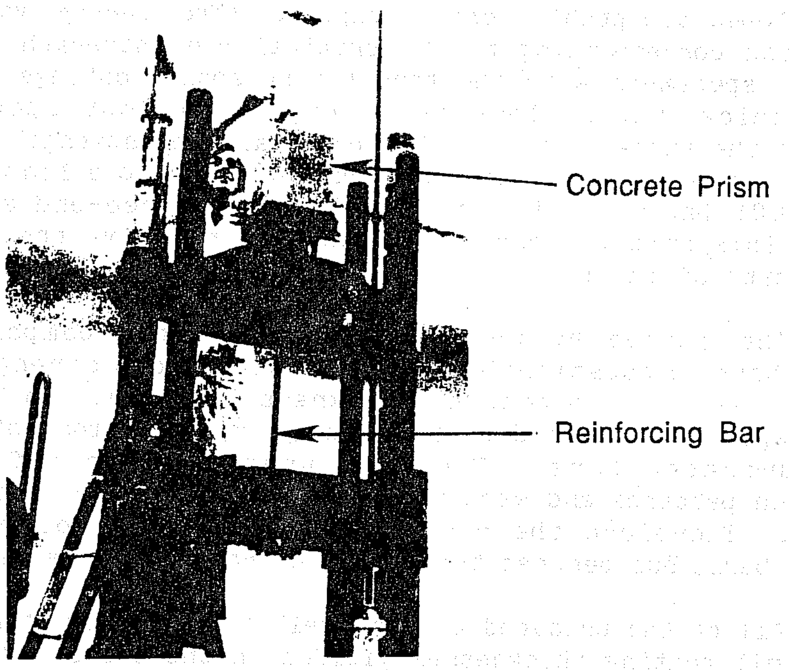


Figure 1.5 Pullout Specimen (Ref. 5)

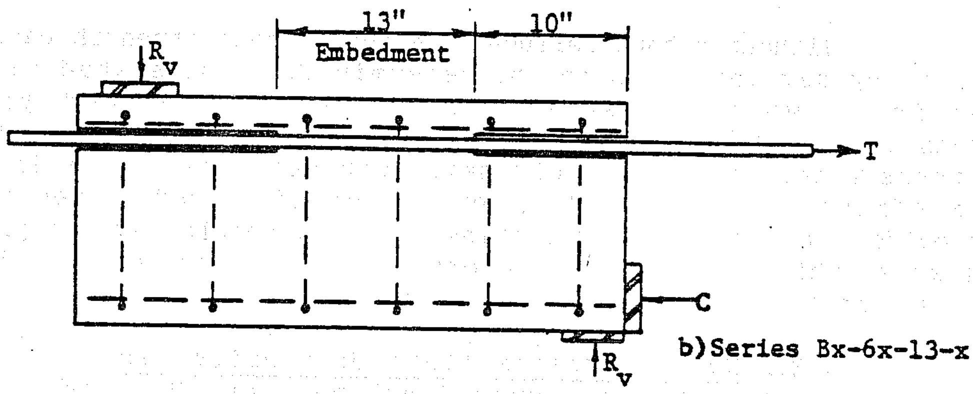


Figure 1.6 Beam End Specimen (Ref. 6)

Results of the slab specimens showed little difference in crack width and spacing, deflections, or ultimate strengths between coated and uncoated bar specimens. The epoxy-coated bar specimens failed at approximately 4% lower loads than the uncoated bar specimens. However, the tests resulted primarily in flexural failures rather than in bond failures so the actual bond strengths could not be measured.

The crack widths and spacings may have been influenced by the way in which the specimens were tested. The specimens were tested basically as simply supported beams. Therefore the moment gradient was very steep and cracks could not form randomly as they would within a constant moment region.

The beam end specimens were flexural-type specimens in which load was applied to the reinforcing bar. The specimens were supported in such a way as to simulate beam behavior (Fig. 1.6). Splitting occurred along the reinforcing bars during the tests but the primary modes of failure were modified because splitting was restrained by transverse reinforcement. Some tests were terminated after yielding of the steel but before a splitting failure occurred. Based on a few tests which ended in a bond or splitting failure, the uncoated bars developed 17% more bond strength than the epoxy-coated bars. This corresponds to the epoxy-coated bars developing about 85% of the bond of uncoated bars. Results of the fatigue tests showed similar results as for the static tests. To account for the reduction in bond strength due to epoxy coating, it was recommended that the development length be increased by 15% when using epoxy-coated reinforcing bars.

CHAPTER 2

EXPERIMENTAL PROGRAM

2.1 Introduction

This study involved 21 beam tests to compare the bond strength of epoxy-coated and uncoated bars in tension. The bond strength was determined by splicing bars in the center of each beam. Also studied was the influence of epoxy coating on member stiffness and on the spacing and width of cracks.

2.2 Scope of Test Program

Epoxy-coated bars were studied under a wide range of variables. The variables were bar size, concrete strength, casting position, and coating thickness.

Specimens were grouped and cast in nine series. In each series a different combination of the above variables was examined, but the only variable within a series was the coating thickness on the bars. These variables are discussed in the following sections.

2.2.1 Coating Thickness Bars were either uncoated with normal mill scale, or had a nominal coating thickness of 5 mils or 12 mils. These values correspond to the minimum and maximum coating thickness allowed by the ASTM Standard Specification for Epoxy-Coated Reinforcing Steel Bars[7]. Each series included a control specimen with uncoated bars, and either one or two specimens with coated bars. Series with two coated bar specimens included one with a nominal bar coating thickness of 5 mils and one with a nominal bar coating thickness of 12 mils. This was to determine what effect, if any, the thickness of the coating had on the bond strength. Series with only one coated bar specimen had a nominal coating thickness of 12 mils.

2.2.2 Bar Size Two bar sizes, #6 and #11, were used so that the behavior of epoxy-coated bars could be examined for the range of bars most commonly used in applications subject to steel corrosion. A primary use of epoxy-coated reinforcing bars is in bridge decks and slabs subject to corrosion due to de-icing salts. Bars used in bridge decks and slabs are commonly #6 bars. Larger bars are routinely used in structural elements located in

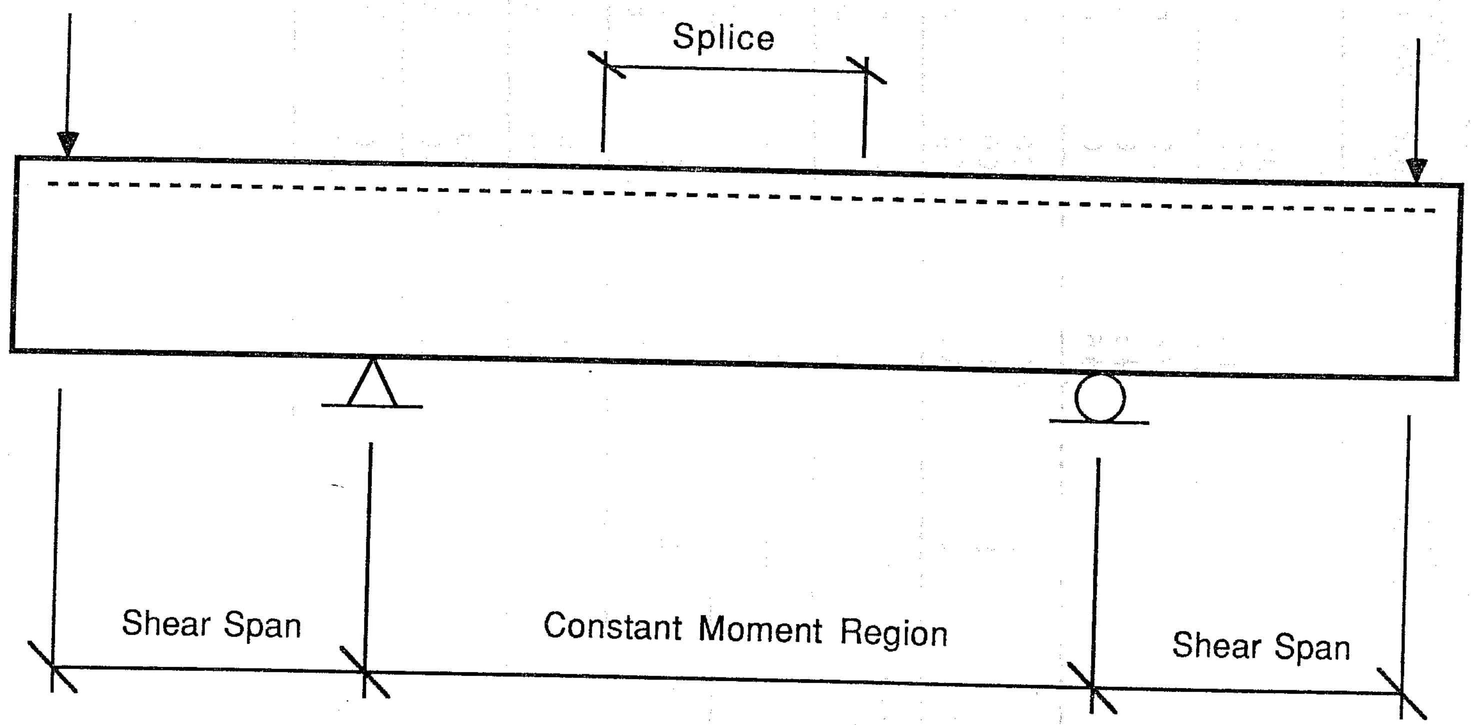


Figure 2.1 Test Setup

2.3 Design of Specimens

In order to study the effect of epoxy coating on the width and spacing of cracks, the loading system was designed to produce a constant moment region in the middle of the specimen (Fig. 2.1). Such a loading produces the most severe splice condition. This also allowed the measurement of crack widths over a region of the beam which was equally stressed.

In the center of the beam, the bars were spliced so that the bond strength could be determined. The splice length was designed so that the bars would not reach yield. If the yield plateau of the steel were reached it would be difficult to make a comparison between tests. Therefore the splice length was designed to develop a steel stress of approximately 45 ksi. The bars had a specified yield strength of 60 ksi.

Current development length provisions in ACI 318[1] do not reflect all the parameters which have been shown to influence development and anchorage. Therefore, the empirical equation developed by Orangun, Jirsa, and Breen [4], described in Chapter 1 was used in designing the splices. Equation 1.1 can be solved by rearranging terms and substituting $d_b f_s / 4\ell_s$ for u (derived in Section 1.2.1):

$$l_s = \frac{d_b[(f_s/4\sqrt{f'_c})-50]}{1.2 + 3(c/d_b)+K_{tr}} \qquad \text{Eq. 2.1}$$

where d_b = bar diameter, in.
f_s = steel stress, psi
f'_c = concrete compressive strength, in.
c = minimum cover or 1/2 clear spacing, in.
K_{tr} = factor considering transverse reinforcement
$c/d_b < 2.5$

Equation 2.1 is the basis of the proposed development length provisions reported by ACI Committee 408[8]. A factor of 1.3 is suggested for top-cast bars with more than 12 in. of fresh concrete cast below the bar, rather than the current top reinforcement factor of 1.4.

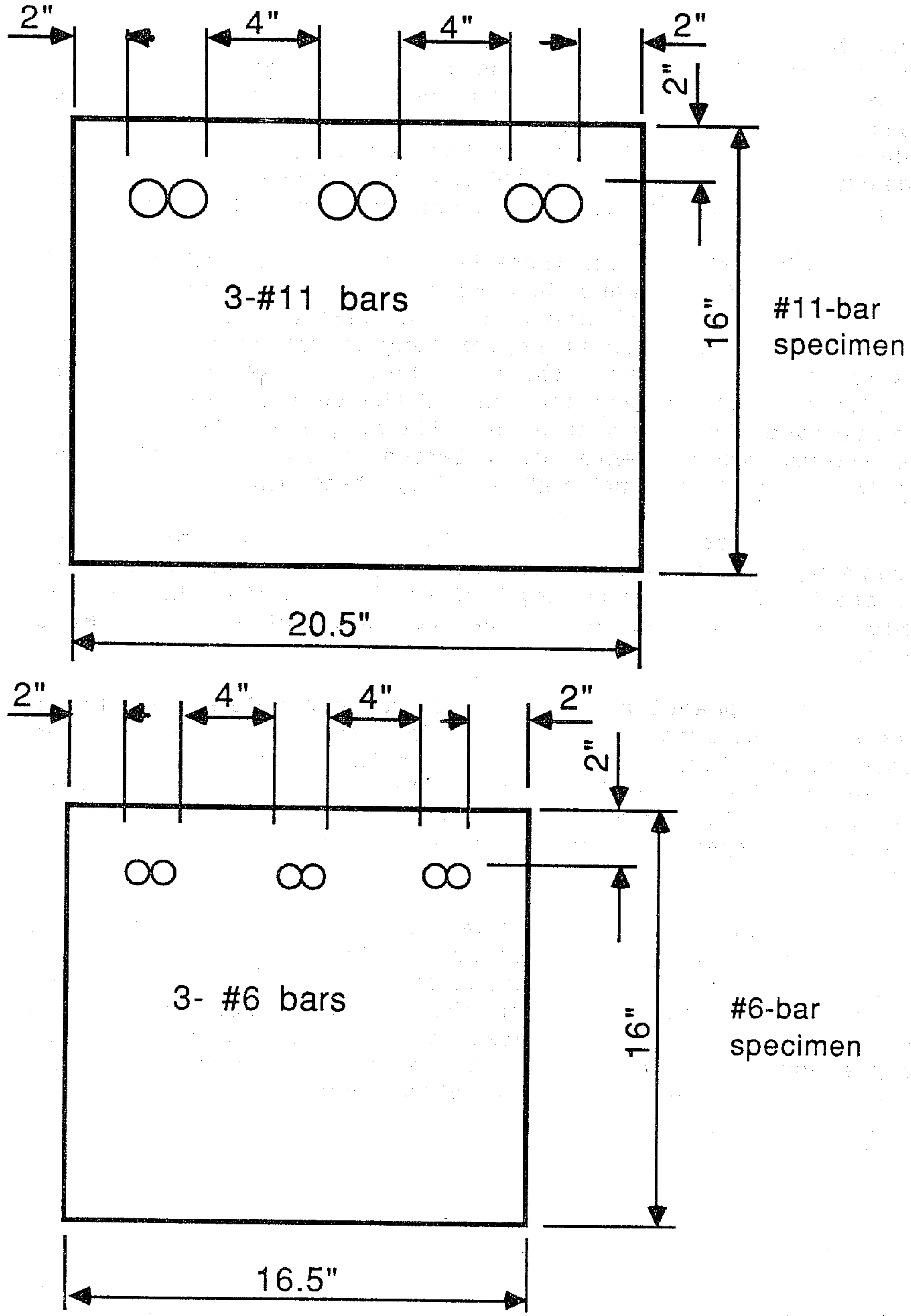


Figure 2.2 Beam Cross Sections

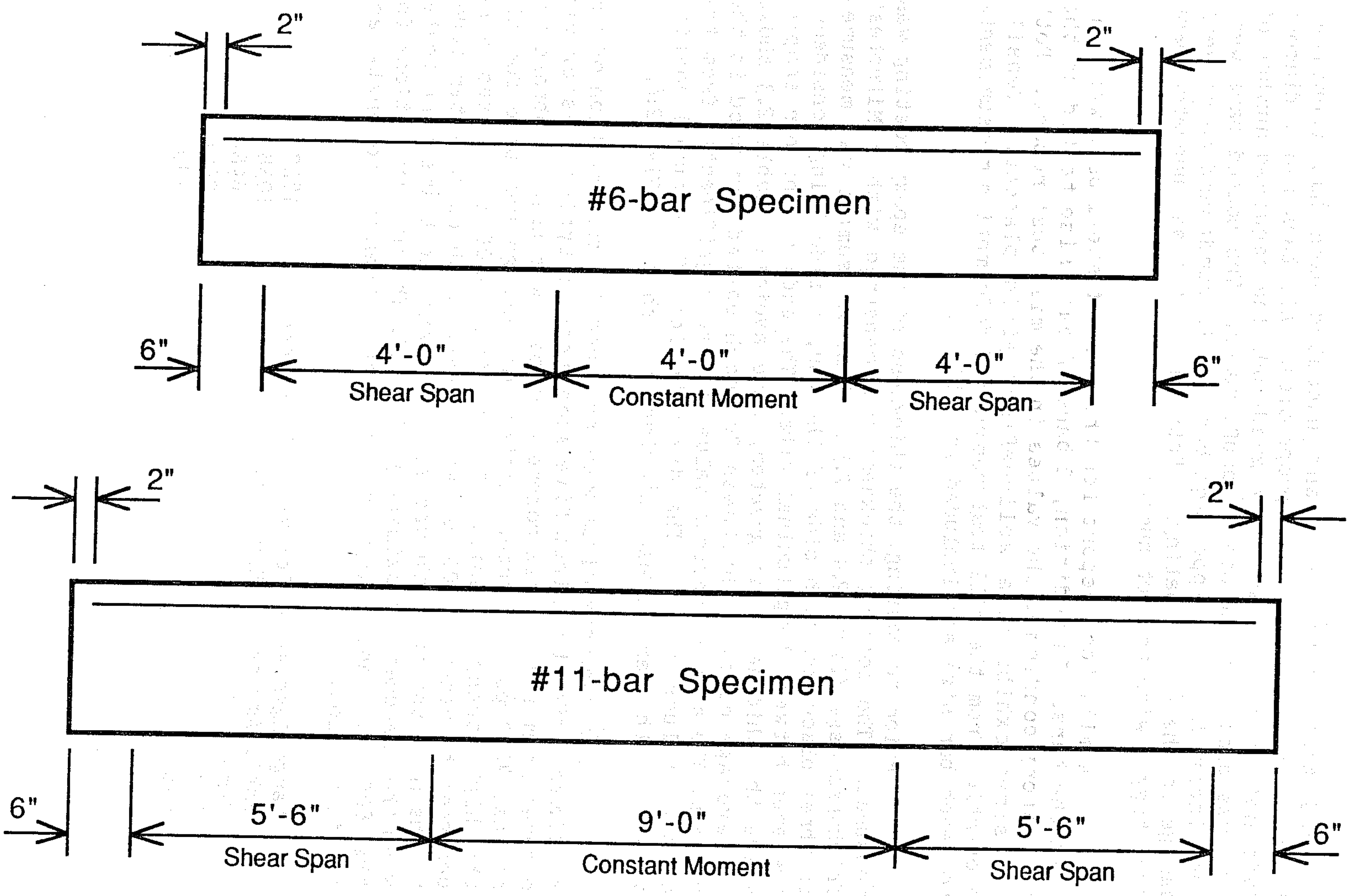


Figure 2.3 Beam Dimensions

Table 2.2 Actual Reinforcing Bar Properties Compared with ASTM A615 Values

Bar Size	#6 Meas.	#6 ASTM	#11 Meas.	#11 ASTM
Maximum Gap *	.189	(.286 max)	.344	(.540 max)
Maximum Spacing	.463	(.525 max)	.759	(.987 max)
Average Height (in)	.106	(.038 min)	.095	(.071 min)
Variation in Weight (%)	3.91	(6 max)	2.6	(6 max)
Yield Strength (psi)	63,300	(60,000 min)	62,800	(60,000 min)
Tensile Strength (psi)	98,640	(90,000 min)	99,680	(90,000 min)
Elongation in 8 inches (%)	11.5	(9 min)	12.3	(7 min)

*Distance between ends of deformation on opposite sides of bar. If ends terminate in longitudinal rib, the width of rib is considered to be the gap.

Table 2.3 Thickness of Epoxy Coating

Specimen	Average Thickness (mils)	Standard Deviation (mils)
12-6-4	10.6	2.0
5-6-4	4.8	2.1
12-6-4r	9.0	2.1
5-6-4r	4.5	1.4
12-11-4	9.1	2.8
5-11-4	5.9	1.9
12-11-4b	11.0	3.9
12-6-8	14.0	3.3
12-11-8	7.4	2.4
12-6-12	10.3	3.3
12-11-12	9.7	2.5
12-11-12b	8.7	2.6

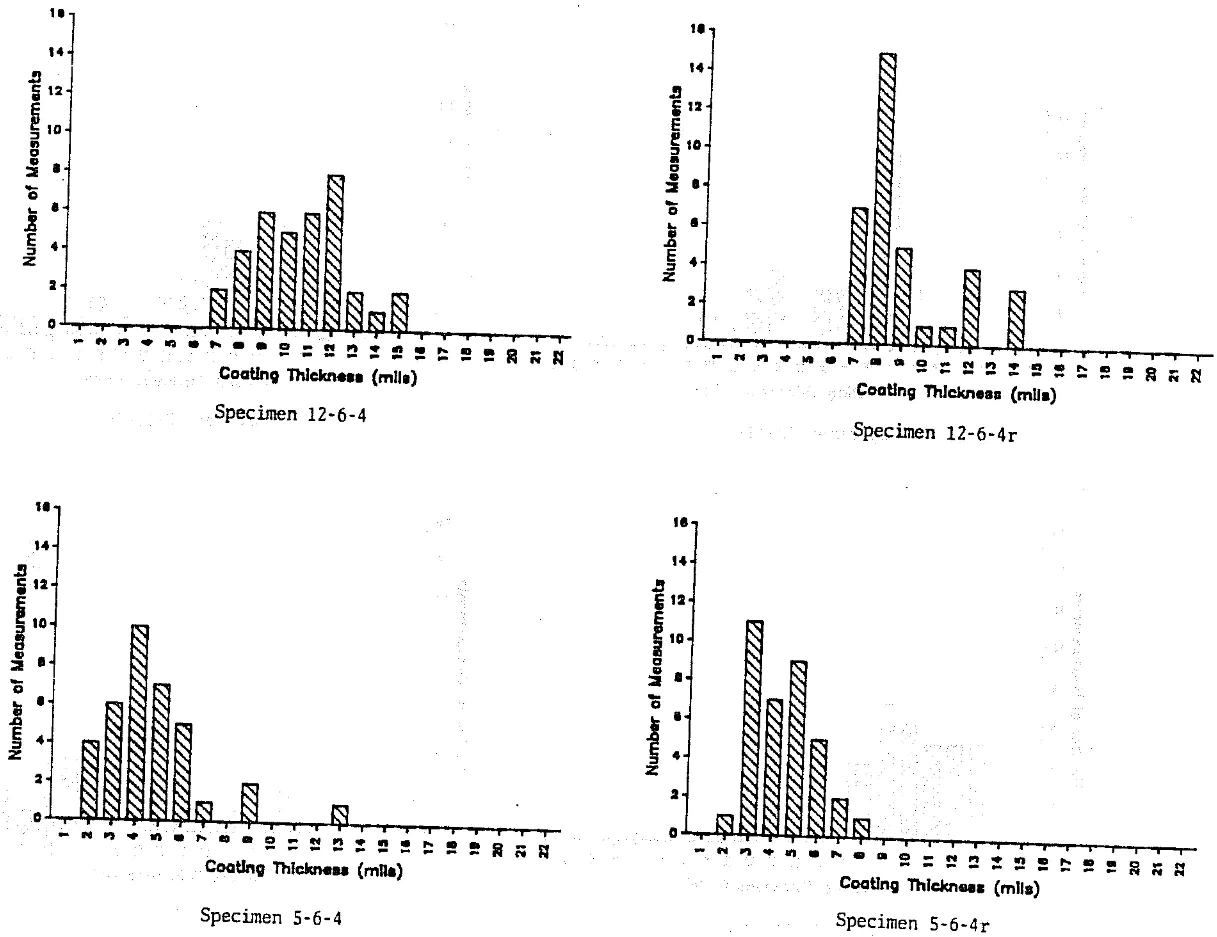
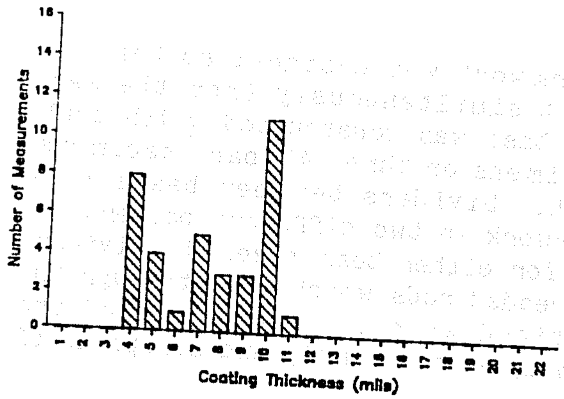
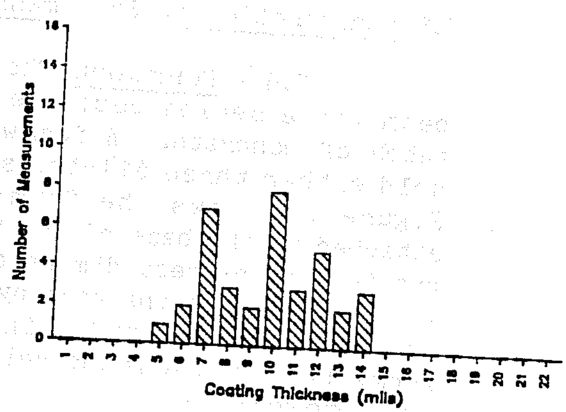


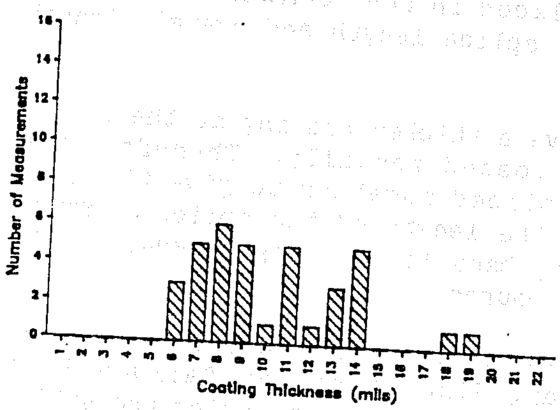
Figure 2.6 Distribution of Coating Thickness Measurements



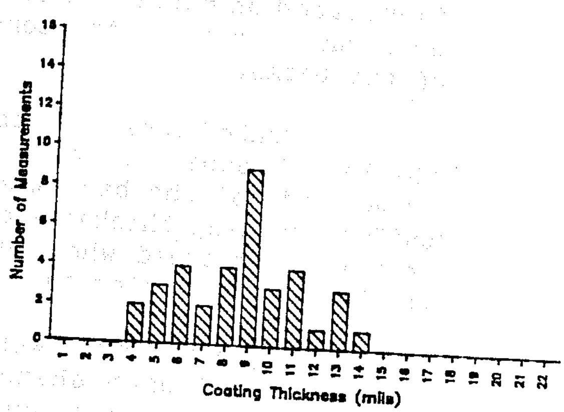
Specimen 12-11-8



Specimen 12-11-12



Specimen 12-6-12



Specimen 12-11-12b

Figure 2.6 Distribution of Coating Thickness Measurements (continued)

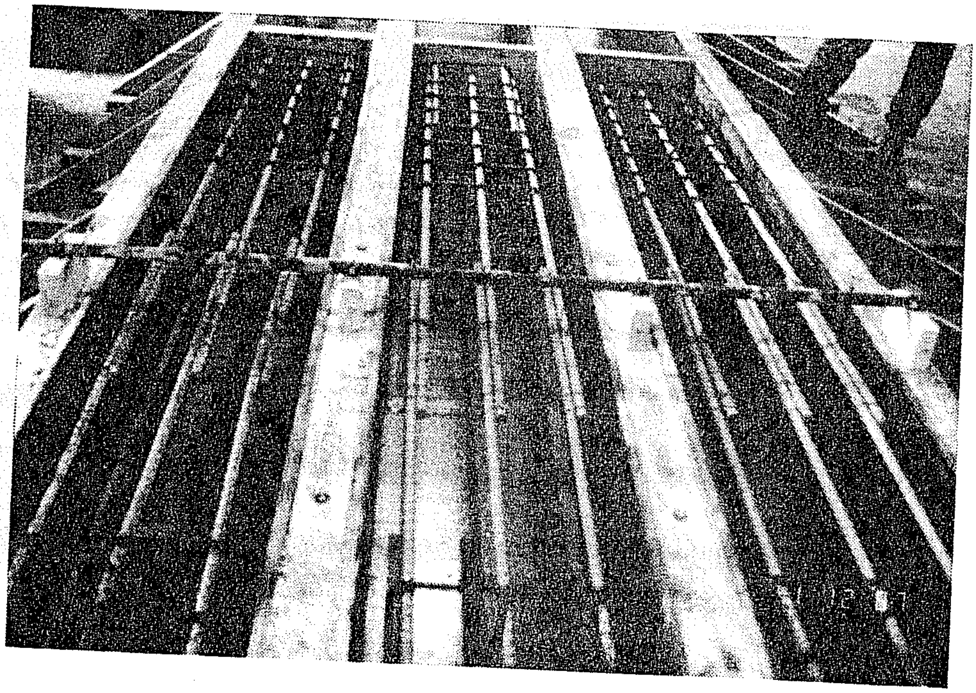


Figure 2.7 Formwork

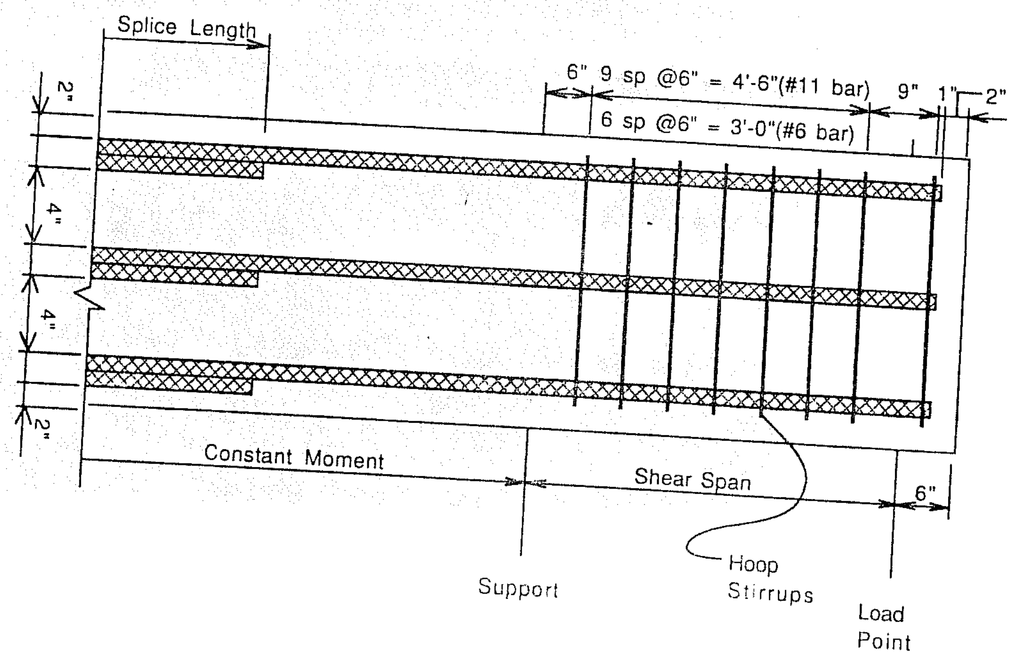


Figure 2.8 Steel Layout

placed from a bucket with the overhead crane. Because of a crane malfunction after these series were cast, casting indoors was no longer possible. Therefore the formwork was moved outdoors so that the concrete could be placed directly from the ready-mix truck and the beams could be removed from the formwork with a forklift.

The concrete was placed in two lifts. The bottom lift was placed in each form and compacted with mechanical vibrators. Then the final lift was placed and compacted. The casting procedure insured that the concrete placed in each beam was of the same consistency. Concrete was placed in cylinder molds while the beams were being cast. Figure 2.10 shows placement of the concrete.

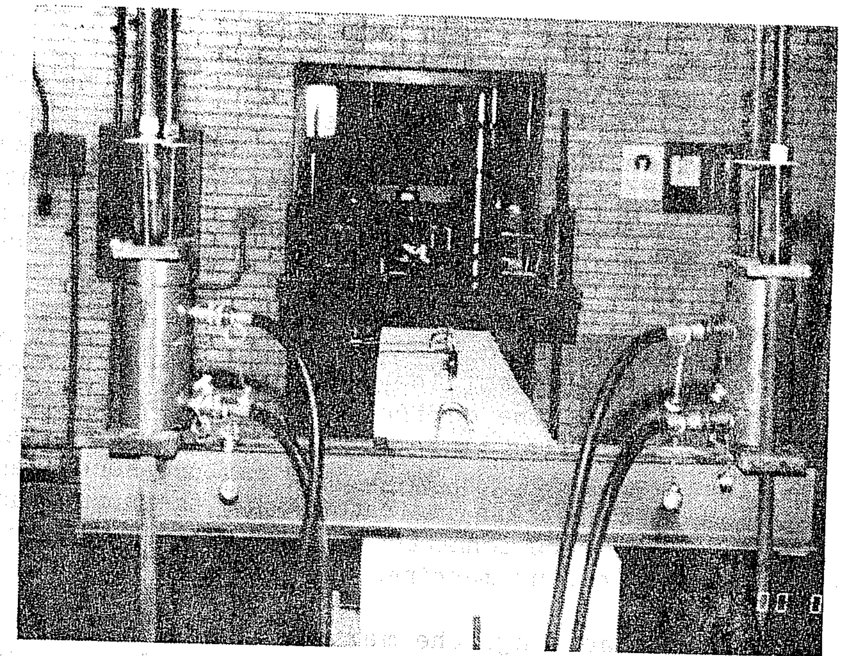
After placing the concrete, the beam surface was finished with trowels. The beams cast indoors were covered with wet burlap and plastic sheets. The beams which were cast outdoors were covered with wet burlap immediately after finishing and were kept soaked during the curing period. The beams were covered with plastic sheets to retain moisture and to prevent shrinkage cracks and volume changes caused by high summer temperatures.

The side forms were usually stripped 1 or 2 days after casting. Beams that were cast indoors were left on the form-base until they were tested. All the specimens that were cast outside were high-strength concrete. The high-strength concrete was designed using a 30% replacement of fly-ash for cement. Fly-ash has a longer hydration period than cement and requires water for a longer period of time for curing. Therefore, the beams were removed from the base after stripping and stored outdoors with burlap, and plastic covering. The burlap was soaked for at least 14 days. Cylinders were stripped on the same day as the beams and cured in the same manner.

2.6 Test Procedure

The test set-up was designed to produce a constant moment region in the middle of the beam, including the length of the splice. To ease marking and measuring of cracks, the beam was tested in negative bending as shown in Fig. 2.1. Figure 2.11 also shows views of the test set-up.

Each specimen was supported by concrete blocks, with 3/4 in. dia. round bars transferring load from the beam to the



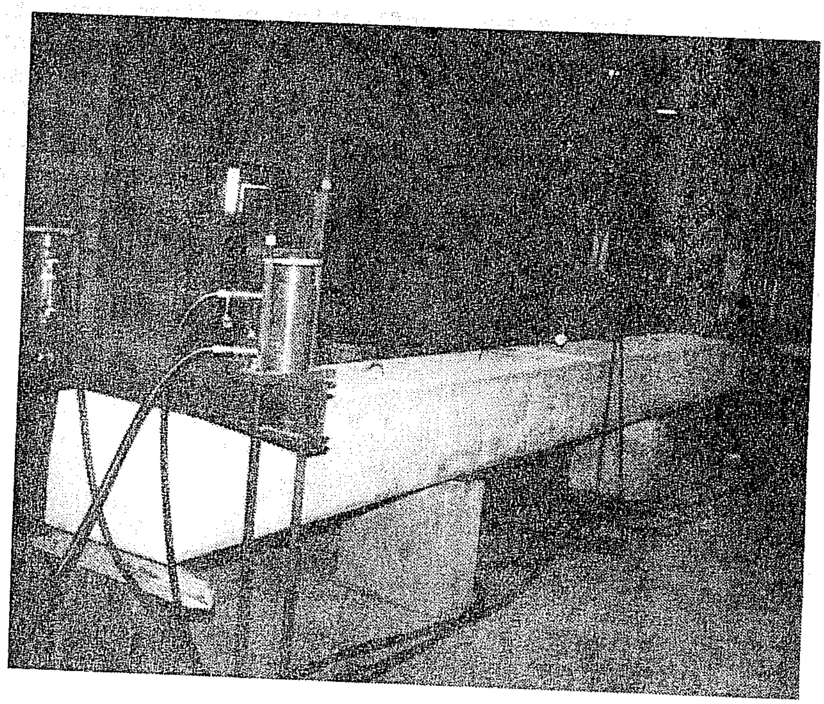


Figure 2.11 Test Setup

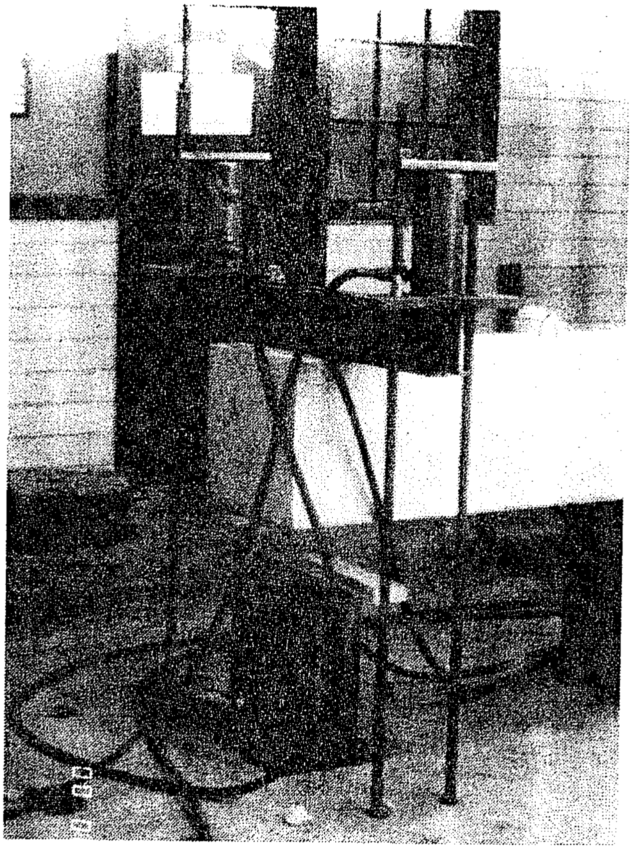


Figure 2.12 Loading System

CHAPTER 3

PRESENTATION AND ANALYSIS OF RESULTS

3.1 Introduction

The results of the 21 beam tests are presented and analyzed. The general behavior of the specimens is discussed in terms of flexural cracking and longitudinal cracking comparing coated and uncoated bar specimens. Based on the results of the tests the performance of coated bars is compared with that of uncoated bars.

3.2 General Behavior

3.2.1 Flexural Cracking Flexural cracks were usually first noticed in the constant moment region outside of the splice. As loading continued, cracks formed along the length of the constant moment region and within the splice. The depth of cracks in the splice region was noticeably less than the depth of cracks outside the splice. At small loads, the bond stress in the splice was well below capacity and there was effectively twice as much steel in the splice region as outside the splice.

In the first series of tests, X-6-4, the location of flexural cracks may have influenced the bond strengths developed. In the coated bar specimen, 12-6-4, flexural cracks formed just inside the length of the splice, but no cracks formed at the ends of the splice. In the uncoated bar specimen, 0-6-4, flexural cracks formed just outside the ends of the splice, but not at the ends. This may have resulted in an effectively longer splice for the uncoated bar specimen than the coated bar specimen and may have influenced the results. Due to the short, 12 in. splice length a small variation in the effective length could have a significant effect.

In order to substantiate the results of the first series of tests, the bars were removed from the tested beams and used to construct a new series of specimens, X-6-4r, with a longer splice. The cover was reduced from 2 in. to 3/4 in. so that the splice would fail at a stress below yield. Since the remaining specimens with #6 bars had also been designed with very short splices, they were redesigned with longer splices to reduce the effect of crack location on the bond strength.

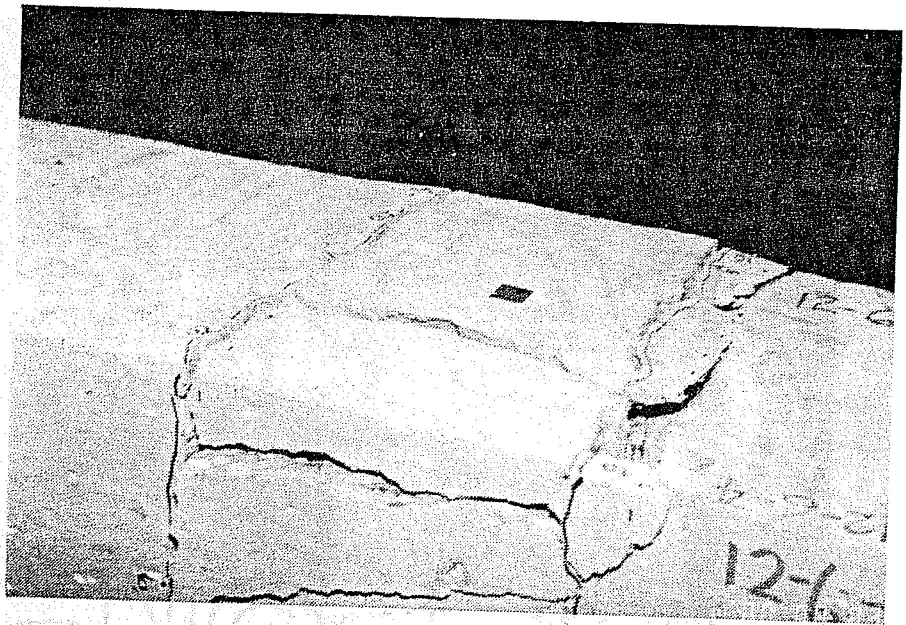


Figure 3.1 Face-and-Side Split Failure

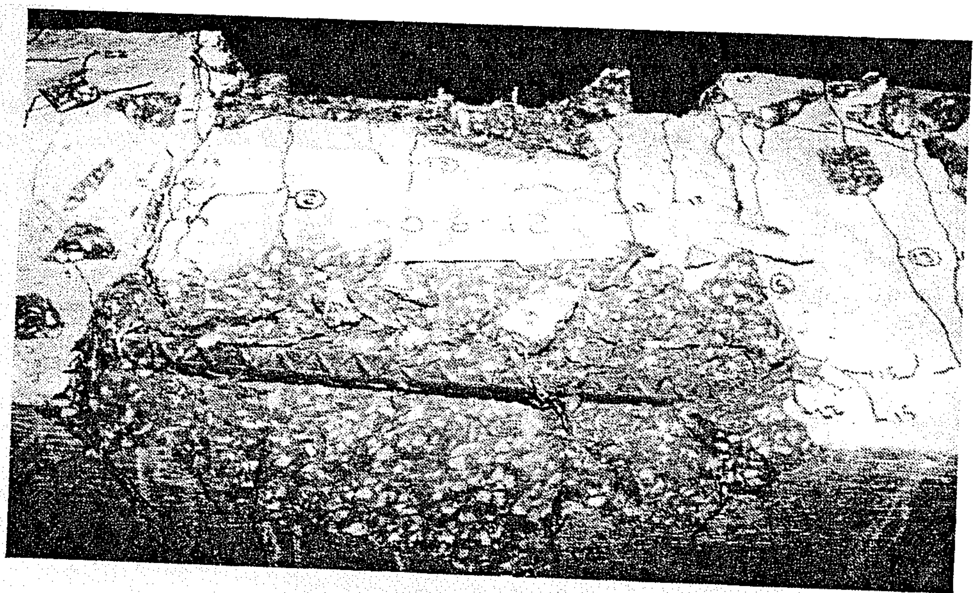


Figure 3.2 V-Notch Failure

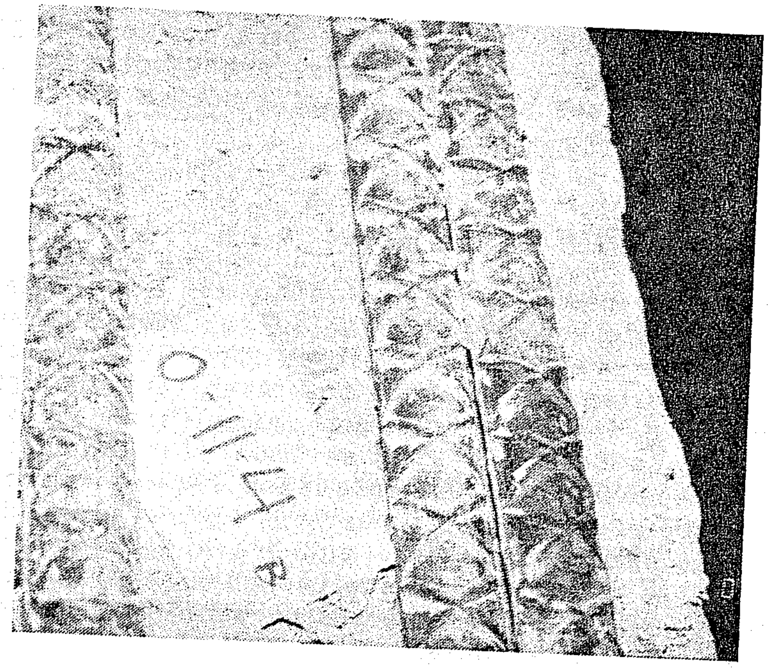


Figure 3.5 Uncoated Bars After Test

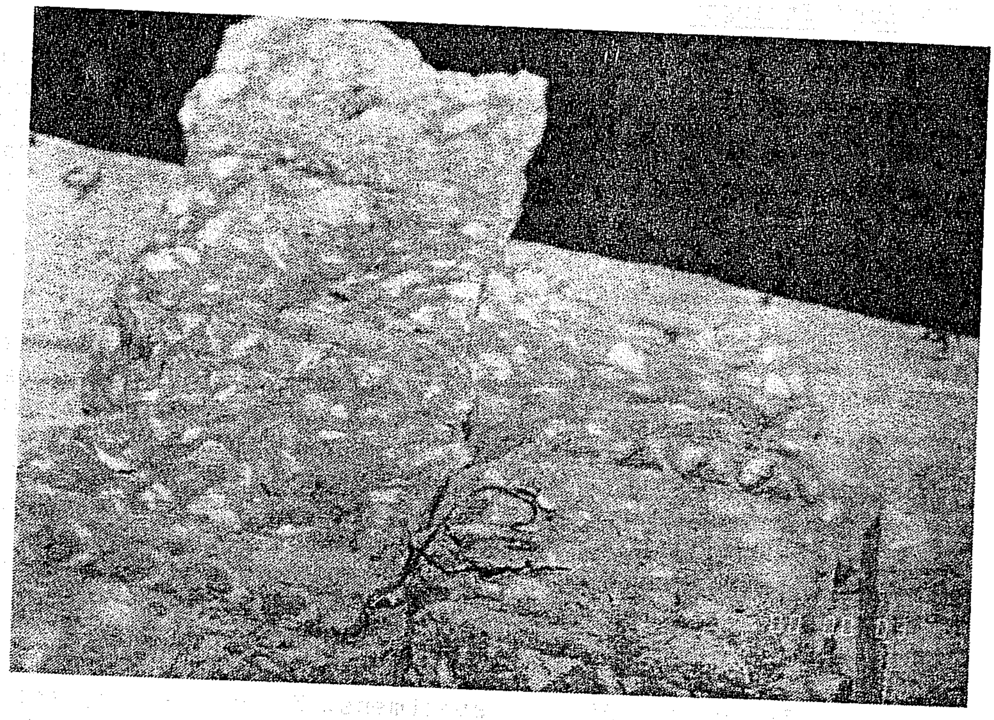


Figure 3.6 Cover on Uncoated Bars After Test

Specimen	l_s (in.)	d_b (in.)	c_b (in.)	c_s (in.)	f'_c (psi)	Coating Thickness (mils)	f_s (ksi)
12-6-4	12	.75	2	2	4250	10.6	33.0
5-6-4	12	.75	2	2	4250	4.8	46.2
0-6-4	12	.75	2	2	4250	0	53.1
12-6-4r	24	.75	7/8	2	3860	9.0	44.8
5-6-4r	24	.75	3/4	2	3860	4.5	47.9
0-6-4r	24	.75	1	2	3860	0	63.3
12-11-4	36	1.41	2	2	5030	9.1	28.3
5-11-4	36	1.41	2	2	5030	5.9	30.4
0-11-4	36	1.41	2	2	5030	0	43.3
12-11-4b	36	1.41	2	2	4290	11.0	24.9
0-11-4b	36	1.41	2	2	4290	0	45.9
12-6-8	16	.75	3/4	2	8040	14.0	35.0
0-6-8	16	.75	7/8	2	8040	0	63.3
12-11-8	18	1.41	2-1/4	2	8280	7.4	25.3
0-11-8	18	1.41	2-1/8	2	8280	0	40.3
12-6-12	16	.75	5/8	2	12600	10.3	41.1
0-6-12	16	.75	3/4	2	12600	0	63.3
12-11-12	18	1.41	2	2	10510	9.7	33.8
0-11-12	18	1.41	2	2	10510	0	46.9
12-11-12b	18	1.41	2	2	9600	8.7	27.5
0-11-12b	18	1.41	2	2	9600	0	43.0

Table 3.1 Actual Specimen Parameters and Measured Test Data

3.2 is based on an equivalent average bond strength and accounts for variation along the length.

Equation 3.2 was used to compute the theoretical bond strength for each specimen. The bond strength using ACI 318 provisions was also determined. The measured bond strength for each specimen was divided by its theoretical or ACI bond strength to obtain a bond efficiency. In order to compare the bond strength of epoxy-coated reinforcing bars to uncoated bars directly, the bond efficiency for each specimen was divided by the bond efficiency of the uncoated bar in the same series. The bond strengths, bond efficiencies, and bond ratio for each specimen, are shown in Table 3.2. Bond efficiency for the coated bars using ACI 318 had a mean value of 0.80 and a standard deviation of 0.17. The mean bond efficiency using theoretical bond strength was 0.69 with a standard deviation of 0.17. The mean bond efficiency of the coated bars was higher when using ACI provisions than when using the theoretical bond strength. For the specimens tested, ACI provisions are conservative. The mean bond efficiency for the uncoated bars using ACI is 1.23 and 0.99 using the theoretical bond strength. Equation 3.2 provides a very accurate estimate of bond strength. Using either bond efficiency to calculate the bond ratio, the mean bond ratio was 0.66 with a standard deviation of 0.07, which is the same as using the ratio of coated to uncoated measured bond strength.

The specimens in which the bars yielded are denoted with a Y next to the measured bond strength. The bond ratio was not significantly influenced by the yielding of the bars. The bond ratio would have been slightly lower for the coated bar specimens in series where the uncoated bars yielded. However, a splitting failure occurred in the splice shortly after the bars began yielding, and indicates that the bond stress required to develop yield was near the strength corresponding to a bond failure.

As shown in Table 3.2, there is a significant reduction in bond due to the epoxy coating. The bond ratio for the coated bar specimens ranged from 0.54 to 0.77. The factors affecting the variation in the bond ratio are discussed in the following sections.

3.4.1 <u>Concrete Strength</u> In order to determine the effect of concrete strength on the comparison between coated bars and uncoated bars, the bond efficiency using theoretical bond strength for each specimen was plotted versus concrete strength in Fig. 3.7. The bond efficiency for the uncoated bars should be close to 1.00. Although some variation is indicated by the lines

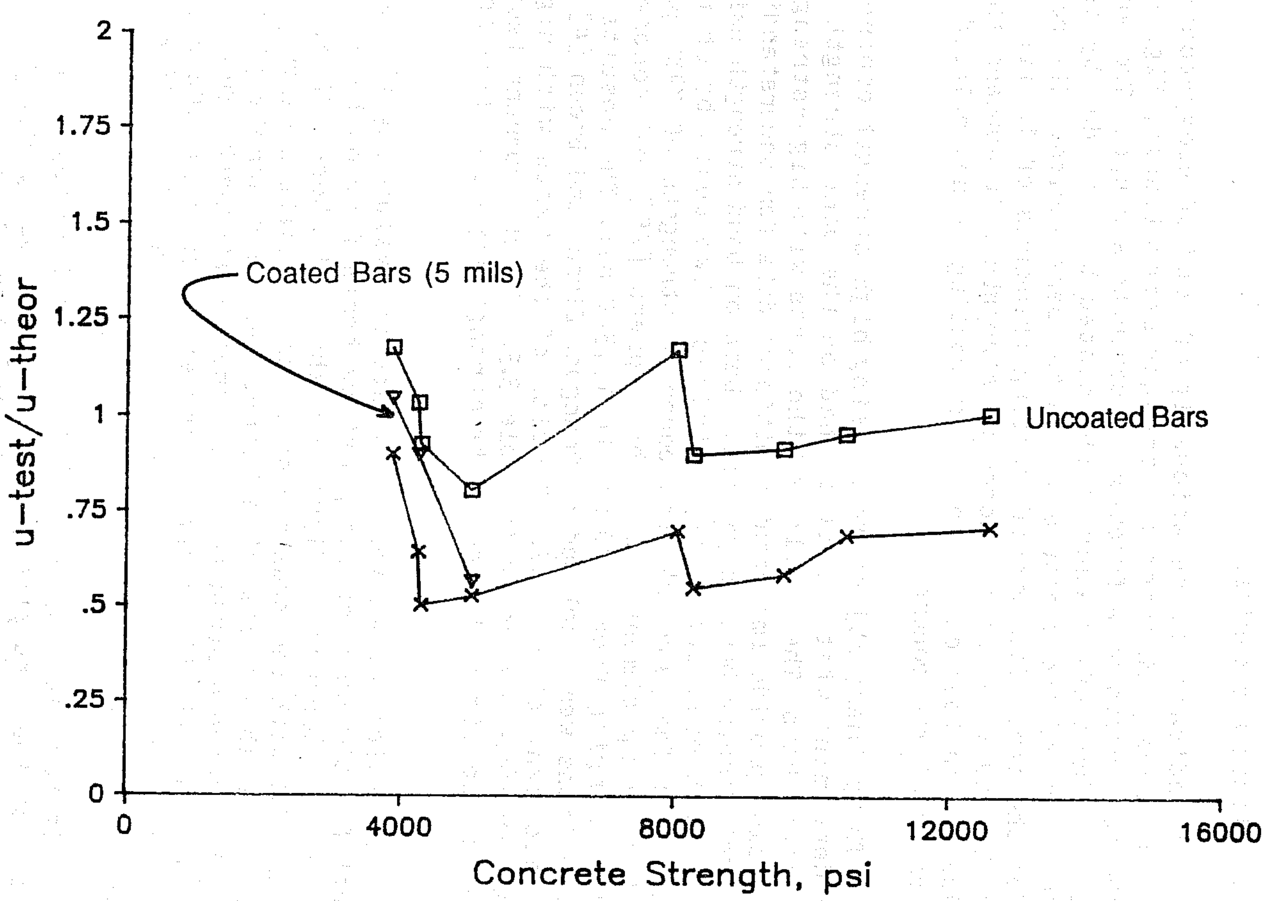


Figure 3.7 Bond Efficiency vs. Concrete Strength

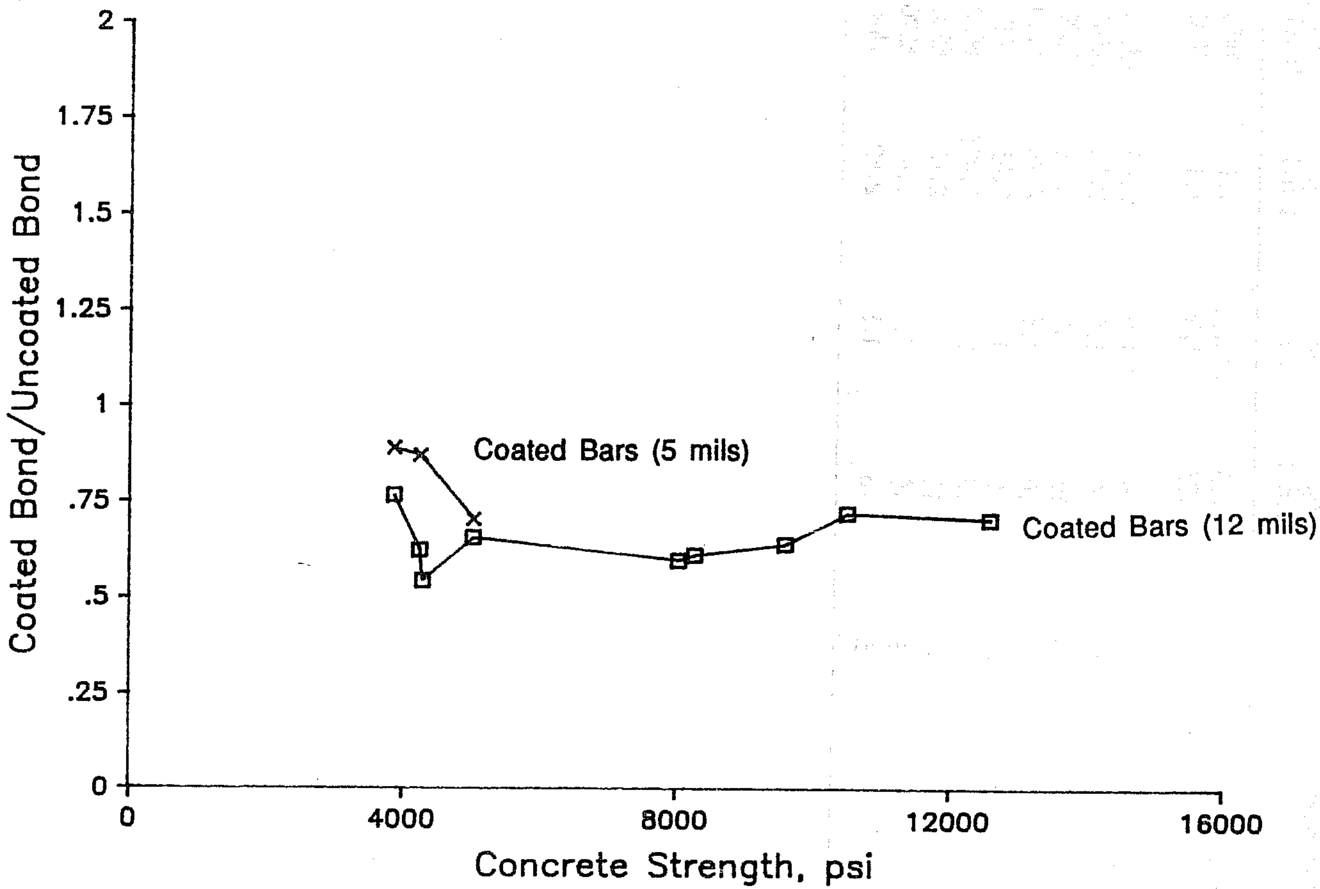


Figure 3.8 Bond Ratio vs. Concrete Strength

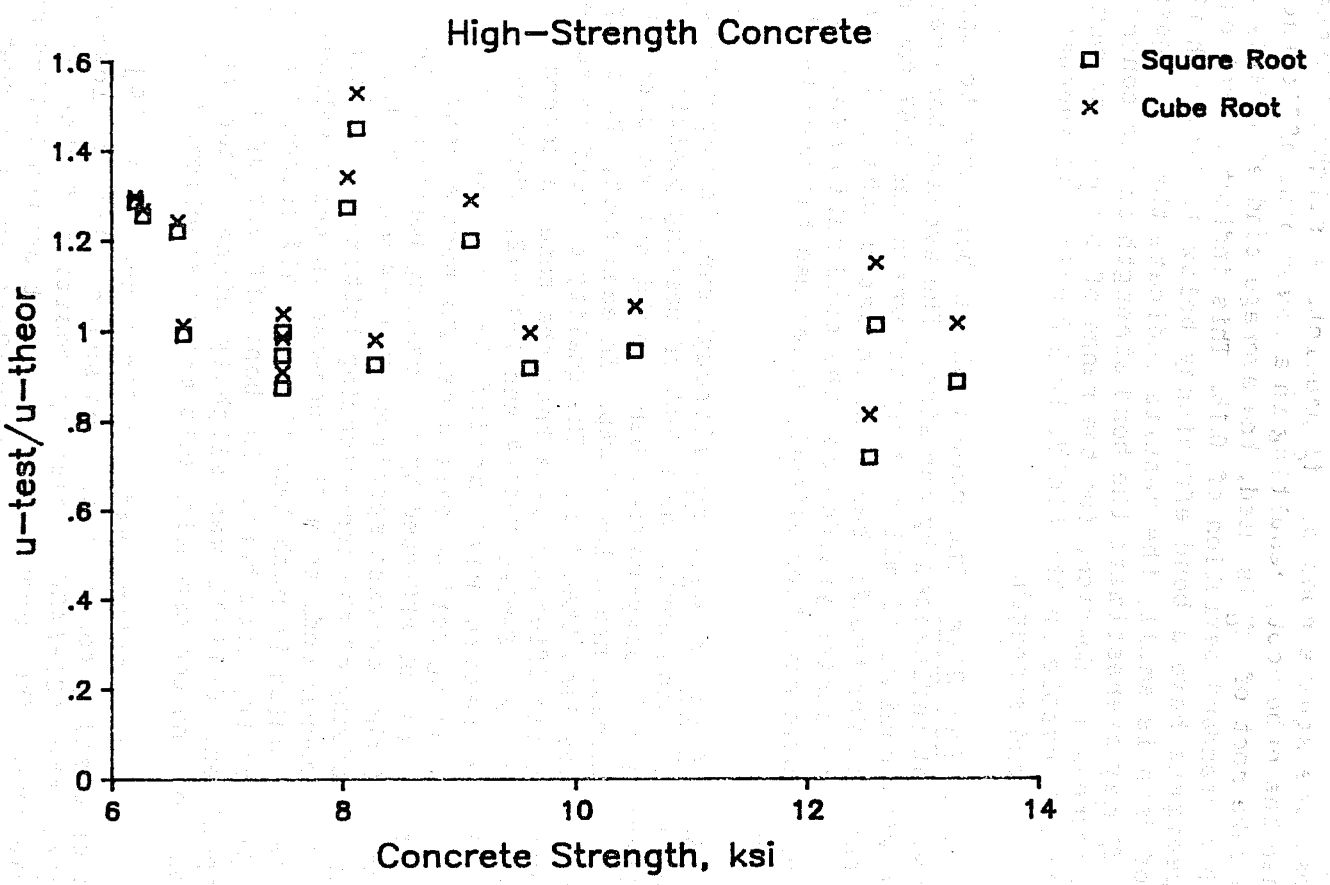


Figure 3.9 Bond Efficiency vs. Concrete Strength

The concrete used in series X-11-12 was high-strength with a slump of 1 to 2 in. before adding superplasticizer. The depth of concrete cast below the bars was approximately 12-1/2 in. Two studies on the effect of superplasticizers on bond [10,11] showed that although the addition of superplasticizers increases the slump, it is not detrimental to the bond between the steel and concrete. Therefore the casting position factor recommended by Breen and Jirsa for this series would also be 1.06.

Since no appreciable loss of bond was seen in the top-cast specimens, the top-cast condition desired was not accomplished. The test results were not normalized with respect to casting position. The purpose in testing both top-cast specimens and bottom-cast specimens was to determine if the epoxy coating would reduce bond beyond the reduction resulting from casting position. Since the bond was not significantly reduced by the casting position, it is difficult to draw conclusions about the additional reduction due to the epoxy coating.

3.4.4 <u>Coating Thickness</u> Figure 3.8 shows that the bond ratio for each of the bars with nominal 5 mil coating thickness is greater than the ratio for the bars with nominal 12 mil coating thickness in the same series. This would indicate that the bond reduction is less for a smaller coating thickness. The actual coating thicknesses, however varied significantly from the nominal values of 5 and 12 mils as can be seen by the distribution of coating thicknesses in Figs. 2.6.

In order to obtain a more accurate relationship between coating thickness and bond reduction, the bond efficiency using theoretical bond strength was plotted versus the average coating thickness in Fig. 3.10. Although the bond efficiency varies widely at any coating thickness, a general decrease in the bond efficiency can be seen as the average coating thickness increases. The coated bars have a much lower bond efficiency than the uncoated bars. The points corresponding to the uncoated bars are grouped about a bond efficiency of 1.0. The majority of the points corresponding to the coated bars are below a bond efficiency of 0.75.

The bond ratio for the coated bar specimens was plotted against the average coating thickness of the bars in Fig. 3.11. To show the variation in the coating thickness, the coating thickness corresponding to one standard deviation above and below the mean was also plotted. The two specimens with the smallest

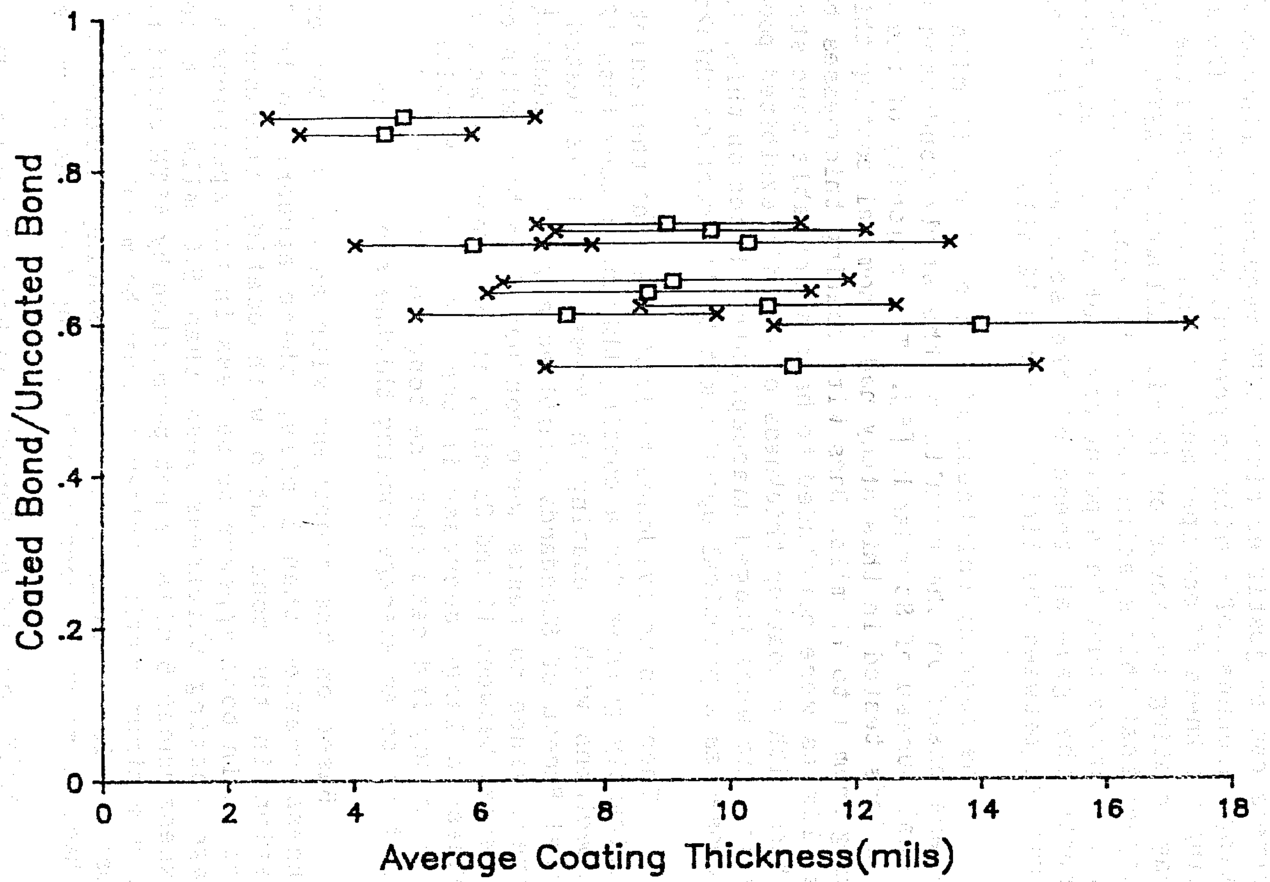


Figure 3.11 Bond Ratio vs. Coating Thickness

mils developed 66% of the bond of uncoated bars. The reduction in bond was very consistent for the range of all variables considered in this study. The average bond ratio was 0.66 with a standard deviation of 0.07.

The results of the tests were consistent, both in the amount of bond reduction due to epoxy coating and in comparison with the theoretical bond calculated using the Orangun, Jirsa, Breen [4] equation, Eq 3.2. This equation forms the basis of the ACI 408 and ACI 318 - Sub B proposals for changes in the Building Code Requirements for Reinforced Concrete [8]. The mean ratio of actual bond strength to theoretical bond strength (bond efficiency) for the uncoated bar specimens was 0.99 with a standard deviation of 0.12. If the specimens with #6 bars at a wide spacing are excluded, the mean bond efficiency is 0.94 with a standard deviation of 0.07. Orangun, Jirsa, and Breen found that the bond strength is increased when bars or splices are widely spaced. The mean bond efficiency of the coated bar specimens was 0.64 with a standard deviation of 0.12. The mean bond efficiency for 54 similar specimens used as the basis of Eq. 3.2 was 1.03 with a standard deviation of 0.12.

3.5 Stiffness

The stiffness of beams with epoxy-coated bars was compared to the stiffness of beams with uncoated bars by plotting the end deflection versus the load for each specimen. The load-deflection curve for each specimen in a series was plotted on the same graph. This allowed direct comparison of the coated bars to the uncoated bars.

Figures 3.12-3.20 show little difference in stiffness between the uncoated bars and the coated bars. The figures also show that the flexural cracking load was not significantly affected by the epoxy coating. In two series, X-6-4r and X-11-12, the coated bar specimens exhibited a greater stiffness than the uncoated bar specimens. This indicates that although some variation existed, it was not biased in favor of uncoated bars.

3.6 Crack Width and Spacing

The cracks outside the splice length are the most accurate representation of the effect of epoxy coating on the spacing and width of cracks. Often, as few as one or two cracks formed between the ends of the splice. Since the constant moment

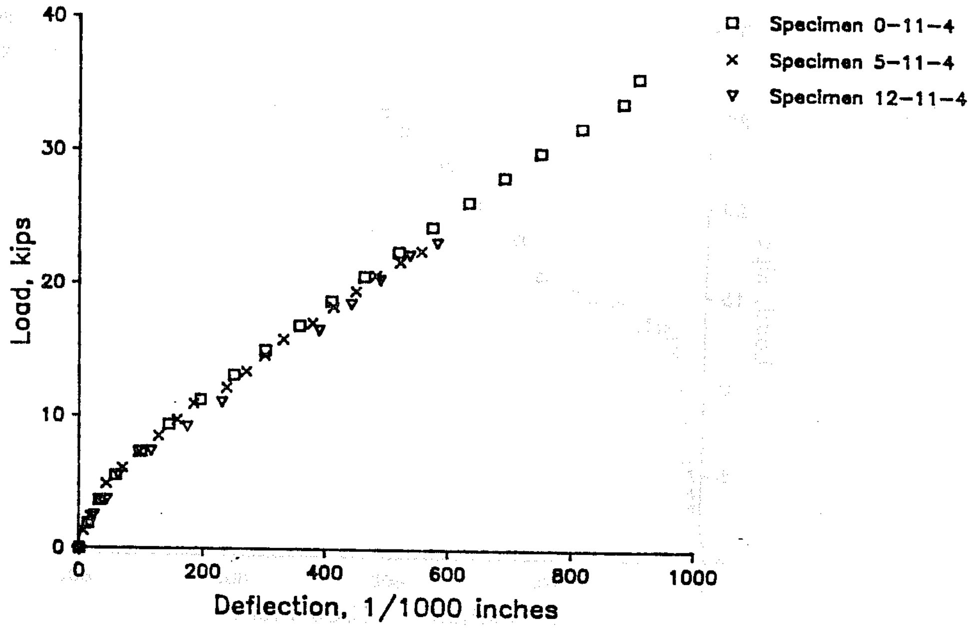


Figure 3.14 Beam End Deflection, Series X-11-4

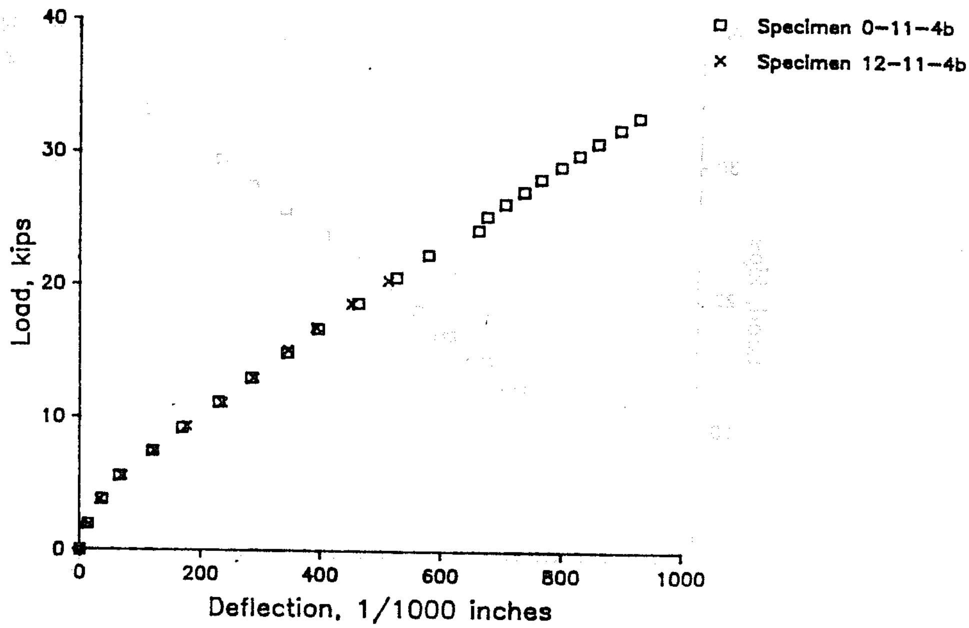


Figure 3.15 Beam End Deflection, Series X-11-4b

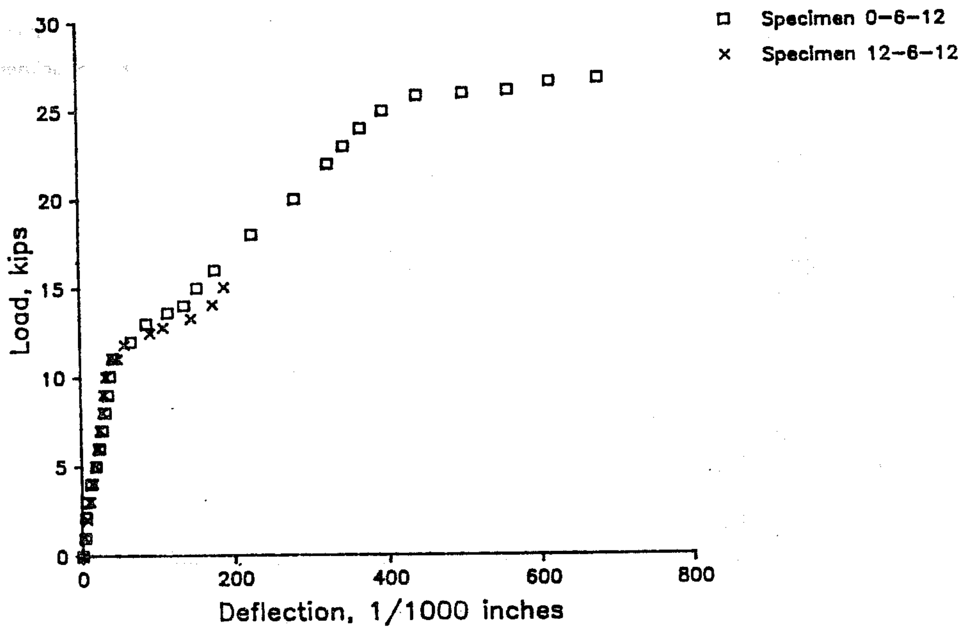


Figure 3.18 Beam End Deflection, Series X-6-12

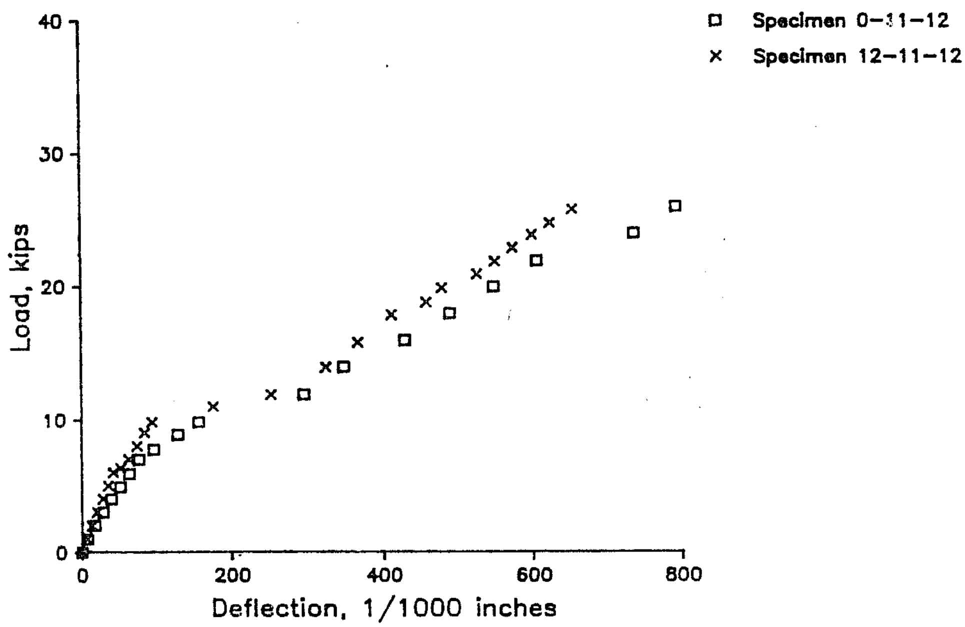


Figure 3.19 Beam End Deflection, Series X-11-12

region outside the splice is longer, more cracks formed and gave a more representative sample for comparing crack spacing. The cracks outside the splice were also much larger than the cracks within the splice which resulted in better accuracy in measuring crack widths. In a structure, flexural cracking in the regions outside of the splice would be of prime concern. Therefore, the constant moment region outside the splice length gave a better indication of the effect of epoxy coating on the spacing and width of cracks.

The cracks outside the splice length were averaged and plotted versus steel stress in Figs. 3.21-3.29. Included in the average were the cracks at the end of the splice. In most tests these had the greatest width, but were not significantly greater than the other cracks outside the splice.

The average crack width is an important parameter because in general, larger cracks allow more corrosive material to reach the bars. However, if bars are coated corrosion will be prevented even though the cracks may be wider.

As a criteria for evaluation, the average crack widths of specimens in each series were compared at a selected steel stress. In most series, this steel stress was around 30 ksi. In some series, however, the coated bar specimens did not develop a steel stress of 30 ksi and the specimens were compared at a lower value. The average crack widths and number of cracks for each specimen are shown in Table 3.4 along with the ratios for coated to uncoated bars. The crack width ratio is the average crack width of a specimen divided by the average crack width of the uncoated bar specimen in the same series. The number ratio is the number of cracks in an uncoated bar specimen divided by the number of cracks in the corresponding coated bar specimen.

In general, the specimens with epoxy-coated bars exhibited wider average cracks than the uncoated bar specimens. Since no difference was seen between deflections of coated and uncoated bar specimens, the total width of all cracks must be equal. As can be seen in Table 3.4, the ratio of average crack widths between coated and uncoated bars is approximately the reciprocal of the number ratio of the number. Specimens with epoxy-coated bars have fewer cracks but the width of the cracks is greater than in uncoated bar specimens.

3.6.1 Coating Thickness Since the number of bars with a nominal coating thickness of 5 mils is limited, it is difficult to establish a relationship between average crack width and

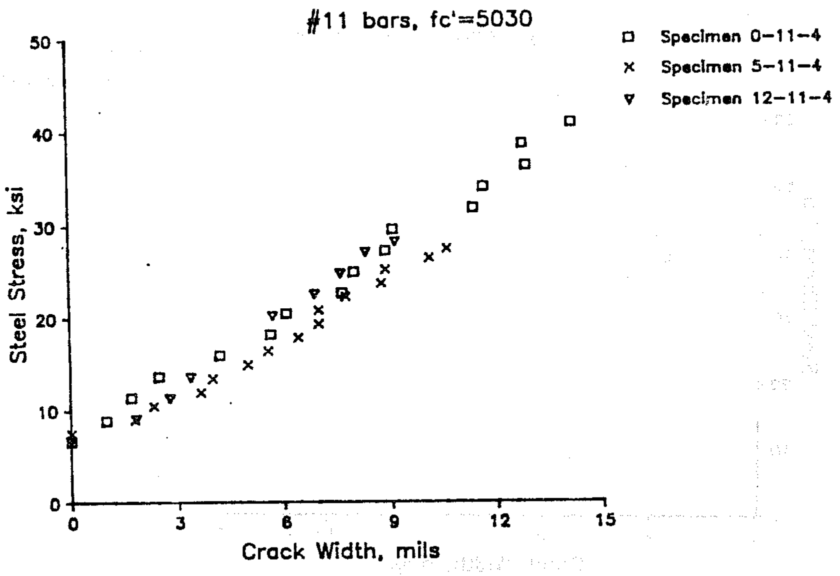


Figure 3.23 Average Crack Widths, Series X-11-4

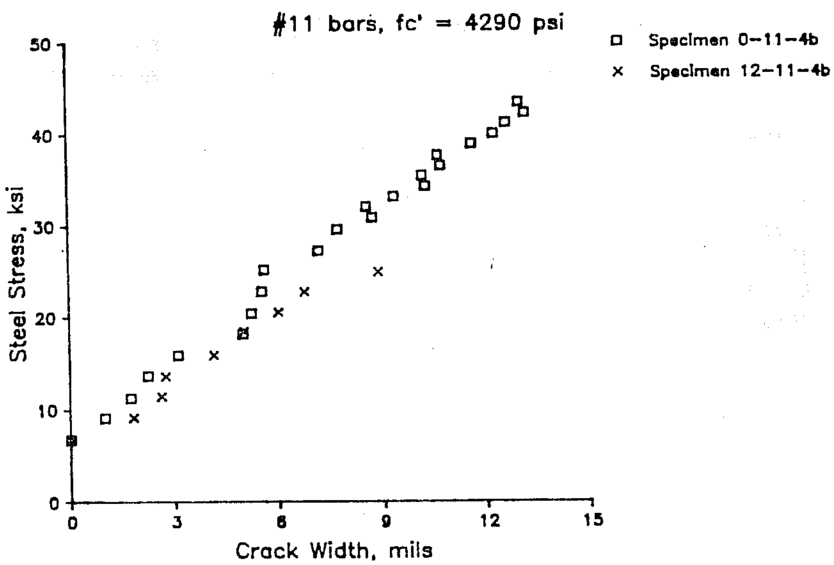


Figure 3.24 Average Crack Widths, Series X-11-4b

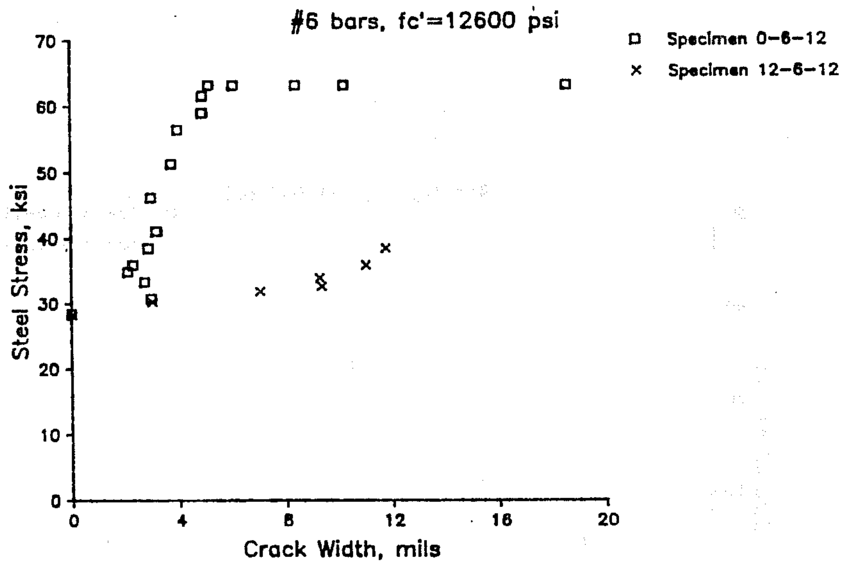


Figure 3.27 Average Crack Widths, Series X-6-12

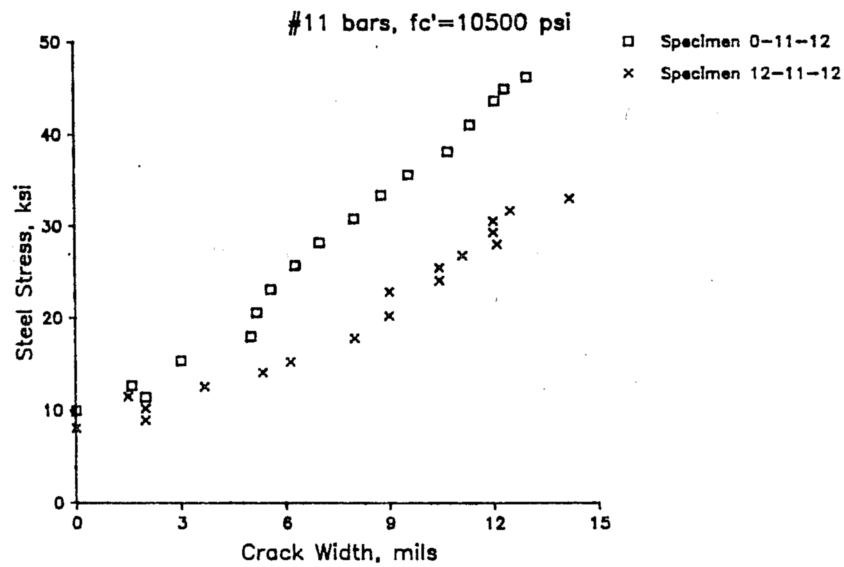


Figure 3.28 Average Crack Widths, Series X-11-12

Table 3.4 Average crack widths

Specimen	Number of Cracks	Number Ratio Coated/Uncoated	Average Crack Width (mils)	Crack Width Ratio Coated/ Uncoated
12-6-4	3	0.5	14	2.3
5-6-4	4	0.67	11	1.8
0-6-4	6	1.0	6	1.0
12-6-4r	6	0.55	5	1.3
5-6-4r	7	0.64	6	1.5
0-6-4r	11	1.0	4	1.0
12-11-4	16	1.14	8	1.0
5-11-4	12	0.86	10	1.3
0-11-4	14	1.0	8	1.0
12-11-4b	11	0.69	9	1.3
0-11-4b	16	1.0	7	1.0
12-6-8	--	---	8	2.7
0-6-8	--	---	3	1.0
12-11-8	12	0.86	10	1.4
0-11-8	14	1.0	7	1.0
12-6-12	6	0.55	12	3.0
0-6-12	11	1.00	4	1.0
12-11-12	12	0.75	12	1.5
0-11-12	16	1.0	8	1.0
12-11-12b	11	0.79	8	1.3
0-11-12b	14	1.0	6	1.0

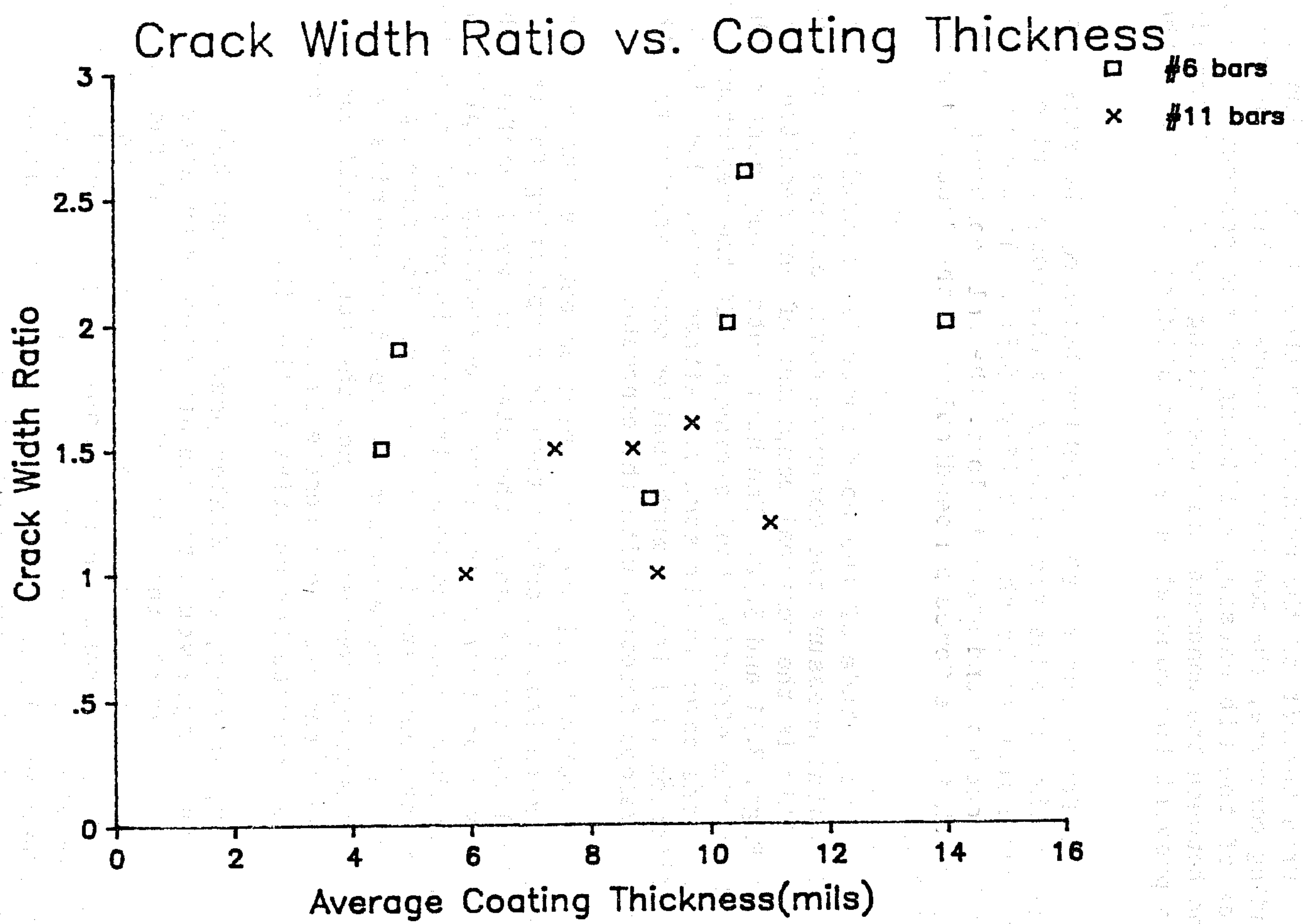

Crack Width Ratio vs. Coating Thickness
#6 bars
#11 bars
Crack Width Ratio
3
2.5
2
1.5
1
.5
0
0
2
4
6
8
10
12
14
16
Average Coating Thickness(mils)

Figure 3.30

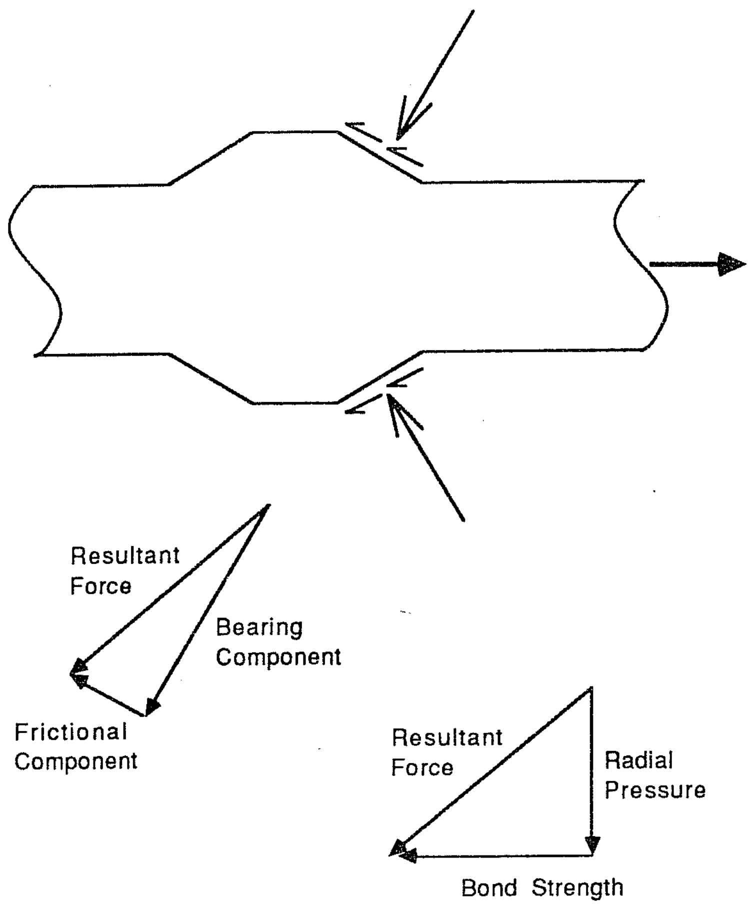


Figure 3.31 Components of Bond
With Friction

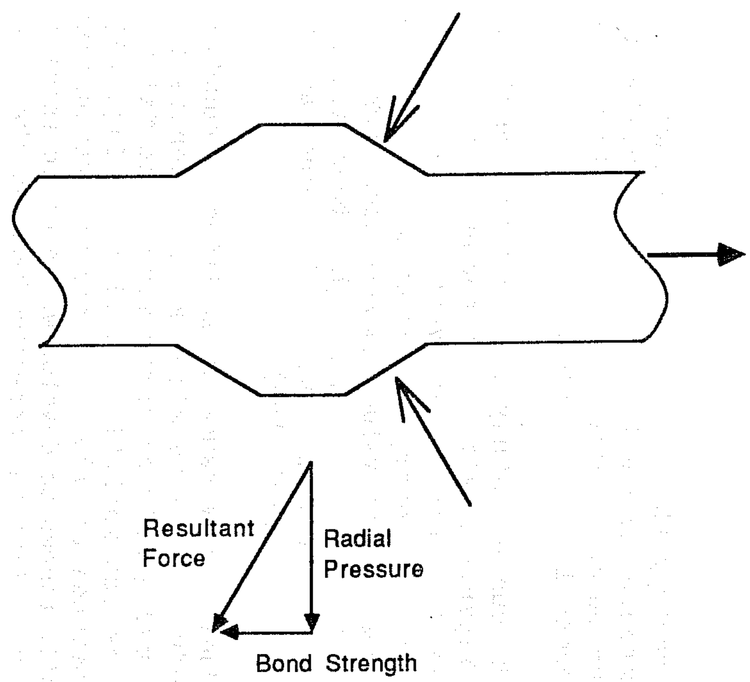


Figure 3.32 Components of Bond
Without Friction

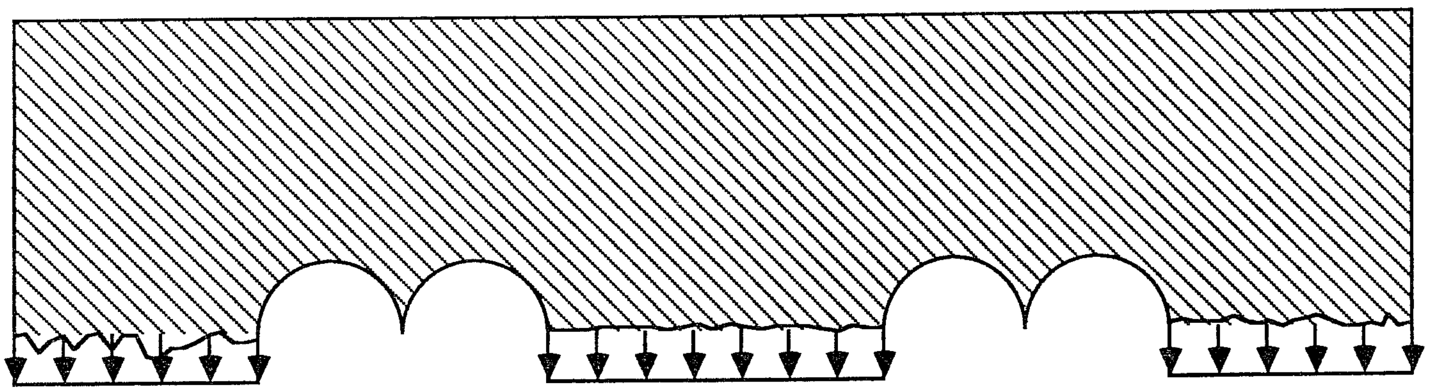


Figure 3.33 Tensile Stresses Across Splitting Plane

CHAPTER 4

DESIGN RECOMMENDATIONS

4.1 Background

Epoxy-coated bars failing as a result of splitting of the concrete cover, developed 66% of the bond of uncoated bars. Based on specimens with an average coating thickness greater than 5.0 mils, the mean bond ratio between coated and uncoated bars was 0.66 with a standard deviation of 0.07.

The tests indicate that the development or splice length must be increased when using epoxy-coated reinforcing bars in typical applications involving limited cover and/or close spacing between bars. The amount of increase is dependent on the type of bond failure which will occur. All of the tests in the current study resulted in a splitting failure. Bond strength comparisons reported in the North Carolina State University study[6], which used stub-beam specimens, showed that epoxy-coated bars confined by transverse reinforcement develop about 85% of the bond of uncoated bars. Comparisons of critical bond strengths in the National Bureau of Standards study[5], which used pullout specimens, showed that epoxy-coated bars develop 94% of the bond of uncoated bars. In the NCSU tests only those which failed in pullout were considered in the strength comparison. Some of the tests in the NBS study were terminated after bars yielded but before a bond failure occurred.

The NCSU study recommended a 15% increase in the development length when using epoxy-coated bars confined by transverse reinforcements. Based on the current study the increase in development length for a splitting failure must be greater than 15%. In order for coated bars to develop the same capacity as uncoated bars, the development length must be increased by the reciprocal of the bond ratio. Based on the average bond ratio (0.66), the development length should be multiplied by 1.5 for epoxy-coated bars where splitting is the mode of failure. If adjustment were based on one standard deviation below the mean bond ratio, the development length factor would be 1.7. However, an increase of this magnitude does not appear to be warranted. If a factor of 1.5 is used to increase the bond strengths developed by the coated bars in the current set of tests, the bond ratios between coated and uncoated bars shown in Table 4.1 are acceptable. Where splitting is prevented by anchorage in mass concrete or by confinement due to a sufficient amount of transverse reinforcement, a much smaller increase (15%) is required. No data are available regarding anchorage of coated bars in compression.

4.2 Proposed Design Recommendations

To account for the influence of epoxy coating on bond and anchorage strength, the following clause is recommended for inclusion in provisions for tensile development and splices.

> Basic development length l_{db} shall be multiplied by the applicable factor when bars are epoxy-coated:
>
> Bars with cover less than $3d_b$ or clear spacing between bars less than $6d_b$........ 1.5
>
> All other cases............... 1.15
>
> The product obtained when combining the factor for top reinforcement with the applicable factor for epoxy-coated reinforcement need not be taken greater than 1.7.

4.3 Comments on Design Recommendations

The magnitude of the development length factor is based on the amount of cover and spacing because a well confined bar will fail in pullout. If the cover or spacing is small, a splitting failure will occur. As noted earlier, the effect of the epoxy coating on bond is less significant when the mode of failure is pullout.

One area which needs to be studied in much greater detail is the influence of transverse reinforcement on the bond strength of epoxy-coated bars. Certainly if the splice or development length is well confined by transverse reinforcement, a splitting failure can be prevented and the effect of the epoxy coating will be small. However, the amount of transverse reinforcement required to provide adequate confinement for epoxy-coated bars is unclear. Generally, both transverse reinforcement and longitudinal reinforcement is epoxy-coated. The confinement provided by coated transverse steel is probably less than that provided by uncoated transverse steel.

specimens in which splitting controls the bond strength represents the condition of most bars in beams and slabs. As the use of epoxy-coated bars increases in all types of structures, tests simulating such installations may be needed to verify the applicability of available data to new conditions. An example is the use of epoxy coated bars in moment resisting frames in seismic zones. Concentric pullout tests, however, have limited application to real structures, mainly for dowel bars into large concrete elements.

CHAPTER 5

CONCLUSIONS

Based on the results of 21 splice tests with epoxy-coated and uncoated bars evaluated in this research study along with data from previous studies, the following conclusions can be made.

1. Epoxy coating significantly reduced the bond strength of reinforcing bars in tension. The amount of the reduction was dependent on the mode of the failure: pullout or splitting.

2. If a splitting failure occurred, the bond strength of epoxy-coated bars was approximately 65% of the bond strength of uncoated bars. If a pullout failure occurred, the bond strength was approximately 85%.

3. The reduction in bond strength was independent of bar size and concrete strength.

4. The reduction in bond strength was insensitive to variations in the coating thickness when the average coating thickness was greater than 5 mils and less than about 14 mils.

5. The width and spacing of cracks was significantly increased by epoxy coating. For #6 bars, the average width of cracks was up to twice the width in uncoated bar specimens.

6. Cracking load and deflections were not significantly affected by epoxy coating.

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