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BOND STRENGTH OF EPOXY-COATED REINFORCING BARS

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Report on a Research Project
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ABSTRACT

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.

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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 reinforcement surrounding the bar, concrete strength, and position of the bars when cast.

Two modes of bond failure are commonly recognized: a splitting failure and a pullout failure. In both cases, the main component of bond is the reaction of the bar deformations against the surrounding concrete which is at an angle to the axis of the bar as shown in Fig. 1.1. The component of this reaction perpendicular to the axis of the bar exerts a radial pressure on the surrounding concrete. If the cover on the bars or the spacing between bars is relatively small, this pressure will cause splitting (Fig. 1.2). The restraint against splitting is dependent on the tensile capacity of the concrete across the splitting plane. Additional restraint may be provided by transverse reinforcement crossing the splitting plane.

If the cover and spacing between bars is great enough, or if enough transverse reinforcement is provided, a splitting failure can not develop and a pullout failure will occur or the bar will yield. In a pullout failure, the concrete between bar deformations is sheared from the surrounding concrete (Fig 1.3). The bond strength for a pullout failure is primarily dependent on the strength of the concrete in direct shear. A pullout failure is more likely for small bars or in large bars where the depth of cover is large or transverse reinforcement is provided around the bars. In both splitting and pullout failure modes the contribution of adhesion to the bond between the bars and concrete is ignored.

The 1963 ACI Code (ACI 318-63)[2] computed the bond strength for a splitting failure as $u = 9.5\sqrt{f'_c}/d_b$. The bond strength was considered independent of the depth of cover. The bond strength for a pullout failure was taken as 800 psi. In 1971, the ACI Code (ACI 318-71)[3] requirements were changed to specifying a required development or splice length. The required lengths were based on the same bond strengths outlined above. The current provisions for bond and development use the same basic development length, l_{db} as the 1971 code, where $l_{db} = 0.04A_bf_y/\sqrt{f'_c}$ but not less than $0.0004d_bf_y$. The basic development length, l_{db} was derived from the 1963 provisions for bond strength by equating the bond strength over the surface of the bar to the total force in the bar at yield.

$$u\pi d_b l_{db} = A_b f_y$$

The actual strength of steel is usually greater than the graded strength. To insure a ductile failure rather than a

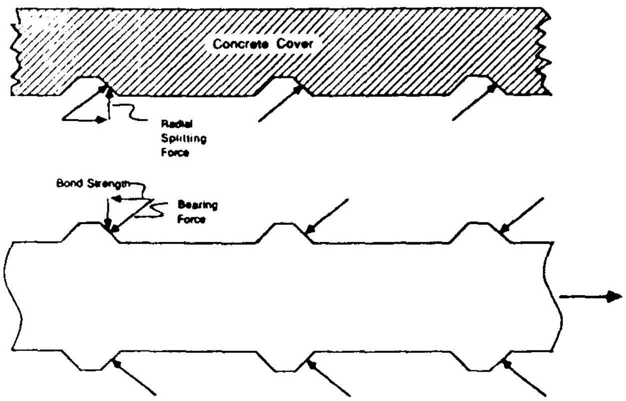


Figure 1.1   Inclination of Bond Stresses

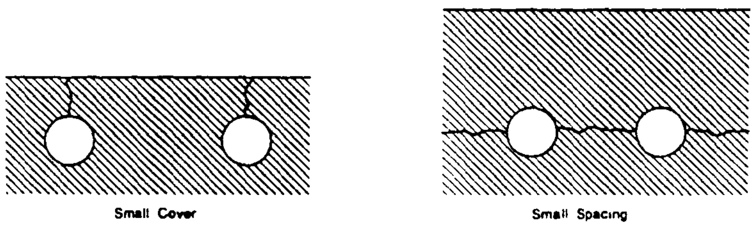


Figure 1.2   Splitting Failure

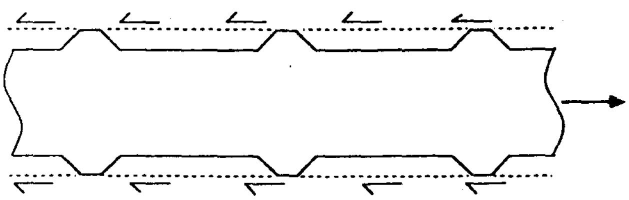


Figure 1.3   Bond Stresses in Pullout Failure

splitting failure, the development length was required to develop 125% of the graded yield strength. If the bond strength for a splitting failure is $u = 9.5\sqrt{f'_c}/d_b$, then:

$$(9.5\sqrt{f'_c}/d_b)\pi d_b l_{db} = A_b(1.25f_y)$$

$$l_{db} = 0.04A_b f_y/\sqrt{f'_c}$$

For a pullout failure the bond strength was taken as 800 psi.

$$(800\text{ psi})\pi d_b l_{db} = A_b(1.25f_y)$$

$$l_{db} = 0.0004d_b f_y$$

1.2.2 Influence of Splitting While the ACI Code value of l_{db} is independent of cover and spacing, it has been recognized for some time that the bond strength is dependent on the depth of cover and the spacing between adjacent bars or splices. For cases where the bond strength is controlled by a splitting failure Orangun, Jirsa, and Breen[4] developed an equation for bond strength which is a function of the thickness of the cover or spacing between bars. The bond strength will be controlled by the lesser of the minimum cover or one-half the clear spacing. As shown in the failure modes for splices in Fig. 1.4, if the side cover, c_s is less than the bottom cover, c_b, the splitting will occur through the side cover and the plane of the splice and will result in a side split failure.

If $c_b < c_s$, the splitting will occur through the bottom (or top) cover. Subsequently, splitting will occur across the plane of the splice and through the side cover, and will result in a face-and-side split failure.

If $c_b << c_s$, splitting will occur through the bottom or top cover and will result in a V-notch failure.

Based on a reevaluation of over 500 available tests on bond, an empirical equation to compute the bond strength was developed by Orangun, Jirsa, and Breen, which accounts for the variation in depth of cover and the spacing between adjacent bars or splices.

$$u/\sqrt{f'_c} = (1.2 + 3c/d_b + 50\, d_b/l_s + K_{tr}) \qquad \text{Eq. 1.1}$$

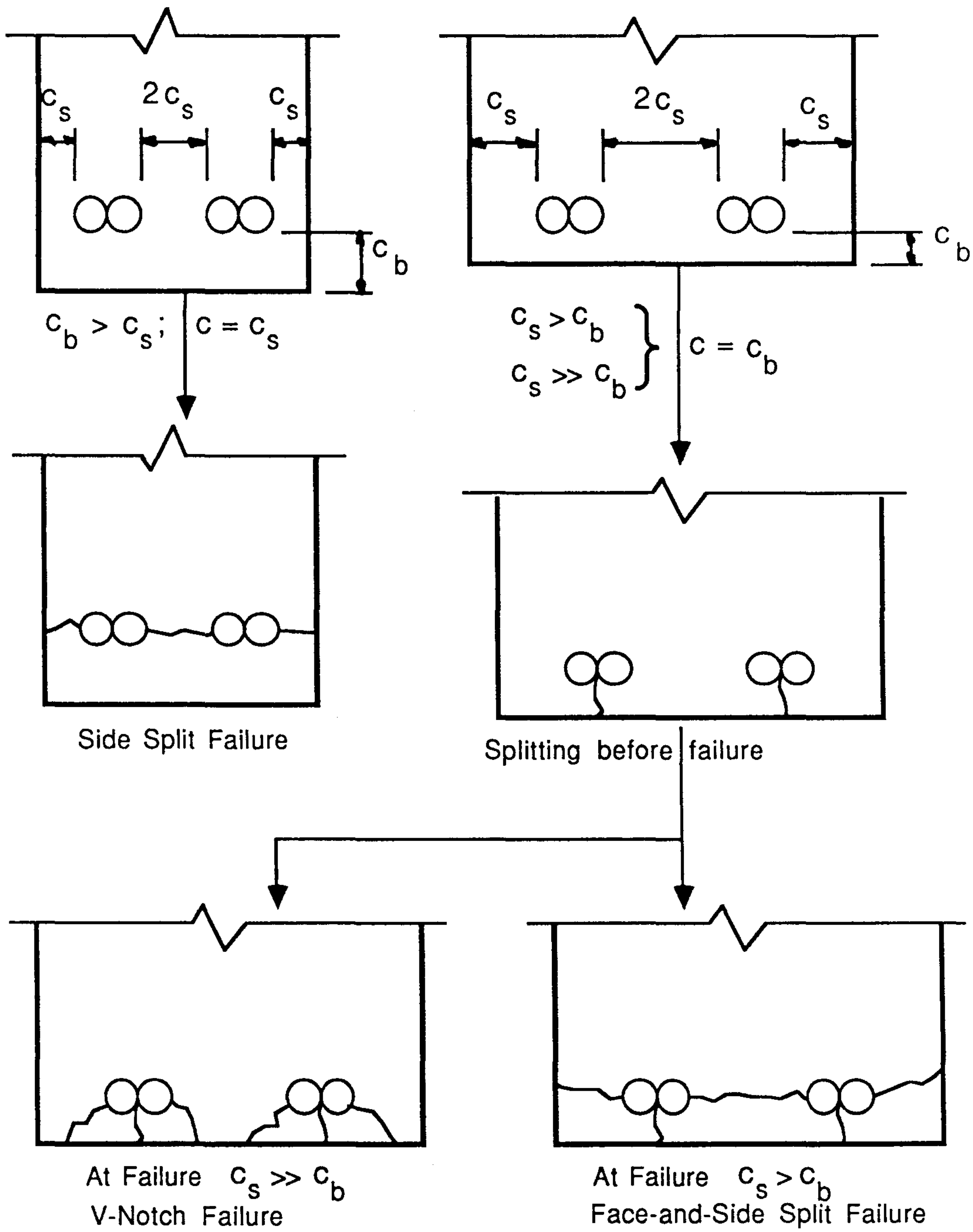


Fig 1.4 Splitting Failure Patterns

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

Equation 1.1 offers a more rational basis for computing bond strength, considering factors which have been shown to influence bond.

Due to the critical nature of developing ductility in structures, the bond between concrete and steel must be insured. Nothing should be applied to reinforcing bars which may decrease the bond capacity. In fact, ACI 318 prohibits any nonmetallic coatings from being applied to reinforcing bars which may decrease the bond capacity (7.4.1).

Epoxy-coated bars, however, have been used for over ten years on the basis that the bond strength is not significantly reduced. Little research has been done on the bond strength of epoxy-coated bars. The two major studies on epoxy-coated bars outlined below found little difference in the bond strength of coated bars compared to uncoated bars.

1.3 Previous Research

1.3.1 National Bureau of Standards Tests The first study on epoxy-coated bars was done at the National Bureau of Standards by Mathey and Clifton[5] and was reported in 1976. A total of 23 epoxy-coated bars with varying thicknesses and different methods of coating application were compared to five uncoated bars in pullout tests. The reinforcing bars tested were all #6 bars. The majority of the coating thicknesses ranged from 1 to 11 mils with two bars having a coating thickness of 25 mils.

The pullout specimens were concrete prisms (10 in. x 10 in. x 12 in.) with a reinforcing bar embedded concentric with the longitudinal axis. Therefore the bars had an embedment length of 12 in. The specimens were tested in a universal testing machine which placed the concrete prism in compression on the face at which the bar was pulled (Fig. 1.5).

Based on the comparison of critical bond strengths, it was concluded that the bars with a coating thickness from 1 to 11

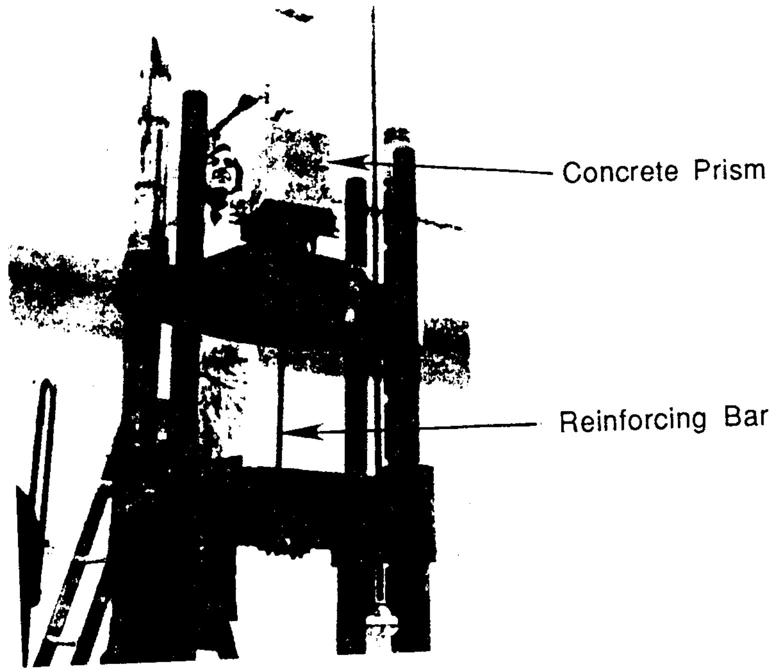


Figure 1.5 Pullout Specimen (Ref. 5)

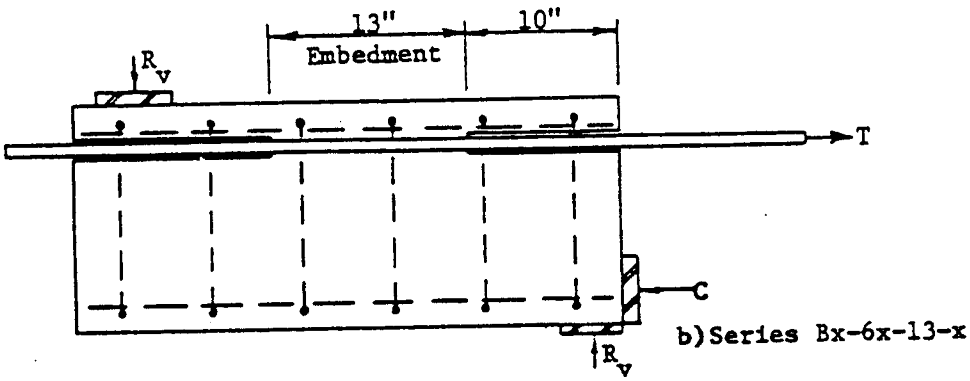


Figure 1.6 Beam End Specimen (Ref. 6)

mils developed acceptable bond strengths. "The average value of applied load corresponding to the critical bond strength in the 19 pullout specimens with the bars having epoxy coatings 1 mil-11 mils thick was 6% less than for the pullout specimens containing the uncoated bars." The critical bond strength refers to the lesser of the bond stress corresponding to a loaded-end slip of 0.01 in. or that corresponding to a free-end slip of 0.002 in. This critical bond strength does not give the actual bond capacity of the bar.

The method by which coated bars were compared to uncoated bars is questionable. The critical bond strengths of the coated bars with a coating thickness from 1 mil to 11 mils were averaged and compared to the average critical bond strength of the uncoated bars. The bars used had two different deformation patterns and were not necessarily from the same heat of steel. Therefore the comparisons made were not between identical bars, but between two groups of randomly selected bars.

All of the uncoated bars as well as the coated bars with 1- to 11-mil coating thicknesses yielded in the tests. Based on this it was again concluded that bars with a coating thickness of approximately 10 mils or less have essentially the same bond strength as uncoated bars. A bond failure occurred in only two of the epoxy-coated bars: those with a coating thickness of 25 mils. Based on this it was recommended that bars with an epoxy coating thickness greater than 10 mils not be used.

Without a bond failure, the actual bond strength capable of being developed can not be determined. As stated in the article, when the stress in the steel exceeded yield considerably, the test was halted. It is not known at what steel stress a bond failure would have occurred. Certainly if the embedment length were long enough or if enough cover were provided, a bar with any coating thickness could develop yield. However this would give no information as to the relative bond strengths between coated and uncoated bars.

1.3.2 <u>North Carolina State University Tests</u> Another study was conducted at North Carolina State University and reported by Johnston and Zia[6] in August 1982. Epoxy-coated bars were compared to uncoated bars with companion specimens under different criteria. Slab specimens were used to compare strength, crack width, and crack spacing. Beam end specimens were used to compare strength under both static and fatigue loadings. The slab specimens contained #6 bars and the beam end specimens contained either #6 bars or #11 bars.

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 marine or other corrosive environments. The same bar sizes were used in a study conducted at North Carolina State University[6] and provided some continuity with previous research.

2.2.3 Concrete Strength Three nominal concrete strengths were used: 4000 psi, 8000 psi, and 12,000 psi. While 4000 psi concrete is commonly used in structures, the use of high-strength concrete is increasing. In addition, research conducted on bond of reinforcing bars in general, using high-strength concrete is very limited. Most research has been done on concrete strengths below 6000 psi and the results have been extrapolated to include high-strength concrete. Therefore tests were conducted using concrete strengths up to a maximum of 12,600 psi.

2.2.4 Casting Position The majority of the specimens were cast with bars in the top position to place the splice under the worst bond condition. The beams were tested with the bars on the top face of the beam to accommodate marking and measuring cracks (Fig. 2.1). Therefore, beams which were top-cast could be tested in the same position as they were cast. Two series were bottom cast to determine the effect of casting position on the bond strength of epoxy-coated reinforcing bars.

Table 2.1 identifies the series of specimens tested. A 3-digit numbering system was used to identify the variables of each beam. The first digit refers to the nominal coating thickness of the bars in mils. The digit 0 means the bars were uncoated. The second digit is the bar size, #6 or #11; and the third digit is the nominal concrete strength in ksi: 4, 8, or 12. The letter, r, refers to a series of specimens which were retested (see Design of Specimens). The letter, b, indicates sets which were cast with bars in the bottom position. As an example, the first beam listed in Table 2.1, 12-6-4, had a coating thickness of 12 mils on #6 bars and a nominal concrete strength of 4 ksi.

Variables were combined to give the most useful information with as few specimens as possible. Each of the three nominal concrete strengths were used in series with #6 bars and #11 bars. In two additional series with #11 bars, the bars were bottom cast; one series with a nominal concrete strength of 4000 psi and one series with a nominal concrete strength of 12,000 psi. All nine series mentioned above contained one uncoated bar specimen and at least one coated bar specimen. Three of the series contained two coated bar specimens.

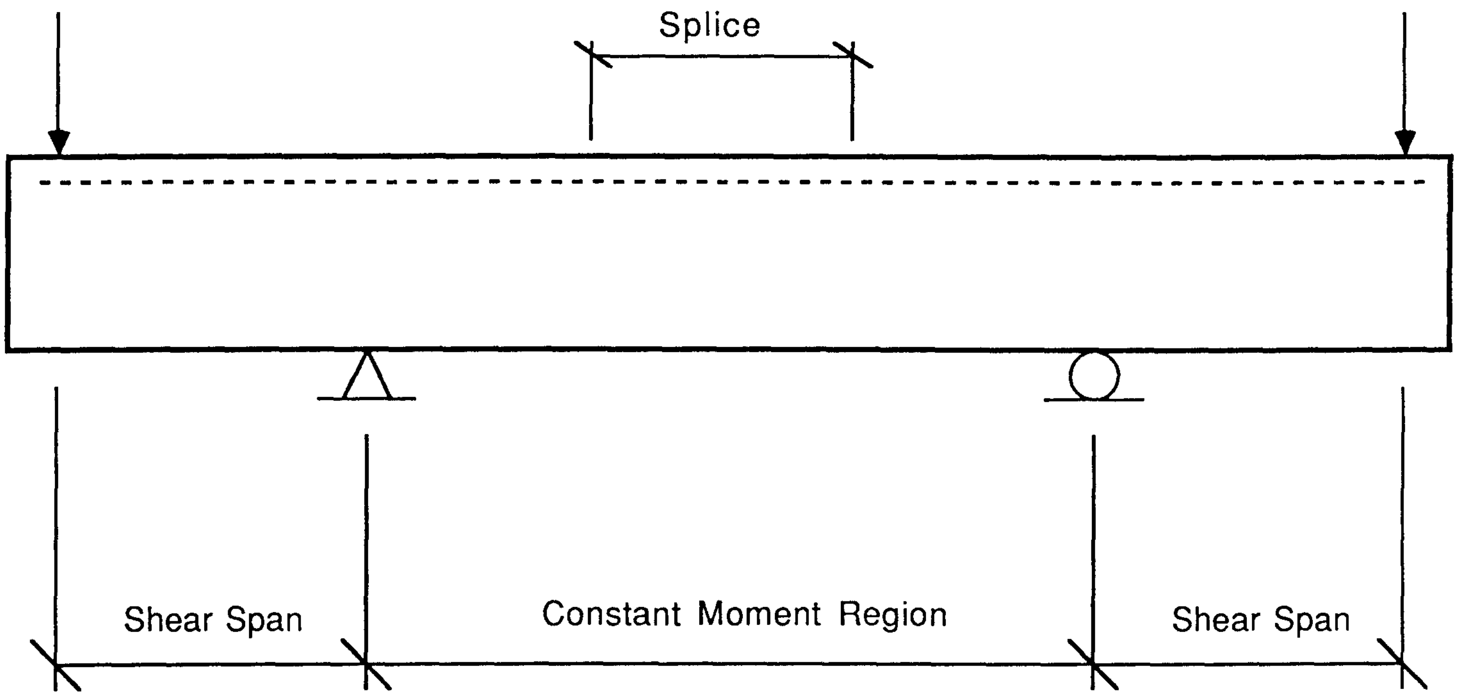


Figure 2.1 Test Setup

Specimen	Nominal Coating Thickness (mils)	Bar Size	Nominal Concrete Strength (psi)	Casting Position
12-6-4	12	#6	4000	top
5-6-4	5	#6	4000	top
0-6-4	0	#6	4000	top
12-6-4r	12	#6	4000	top
5-6-4r	5	#6	4000	top
0-6-4r	0	#6	4000	top
12-11-4	12	#11	4000	top
5-11-4	5	#11	4000	top
0-11-4	0	#11	4000	top
12-11-4b	12	#11	4000	bot
0-11-4b	0	#11	4000	bot
12-6-8	12	#6	8000	top
0-6-8	0	#6	8000	top
12-11-8	12	#11	8000	top
0-11-8	0	#11	8000	top
12-6-12	12	#6	12000	top
0-6-12	0	#6	12000	top
12-11-12	12	#11	12000	top
0-11-12	0	#11	12000	top
12-11-12b	12	#11	12000	bot
0-11-12b	0	#11	12000	bot

Table 2.1 Test Parameters

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.

As an example, for specimen 0-6-4, to develop a steel stress of 45000 psi:

with $d_b = 0.75$ in.
$f_s = 45000$ psi
$f_c' = 4000$ psi
$c = 2$ in.
$K_{tr} = 0$

the splice length is:

$$l_s = \frac{0.75[(45{,}000/4\sqrt{4000}) - 50]}{(1.2 + 3(2.5) + 0)} = 11 \text{ in.}$$

Applying the factor for top-cast bars, a splice length of 14 in. is required to develop 45000 psi in the steel. The actual splice length used in specimen 0-6-4 was 12 in.

In order to have a more representative cross section, 3 bars were used in each beam. With three bars, different failure modes could be developed more realistically.

Following the splitting failure patterns outlined in Chapter 1, the beams in this series were designed so that the side cover was one-half the clear spacing between bars and equal to the top or bottom cover. This meant that the failure could occur as either a side split failure or a face-and-side split failure. A cover of 2 in. on the bars was chosen as a typical depth of cover for beams.

Given the 2 in. cover and corresponding 4 in. clear spacing, beam widths of 16.5 in. and 20.5 in. were chosen for the #6 and #11 bars, respectively. Cross sections of the specimens at the location of the splice are shown in Fig. 2.2. In order to satisfy the condition of having more than 12 in. of concrete below the top-cast bars, an overall depth of 16 in. was chosen. This resulted in approximately 12.5 in. of concrete below the #11 bars and 13.25 in. of concrete below the #6 bars. Using the same depth for both sizes of bars allowed all beams to be cast in the same set of forms.

After the first set of specimens, X-6-4, was tested, the short 12 in. splice length appeared to produce a failure which precluded direct comparison between coated and uncoated

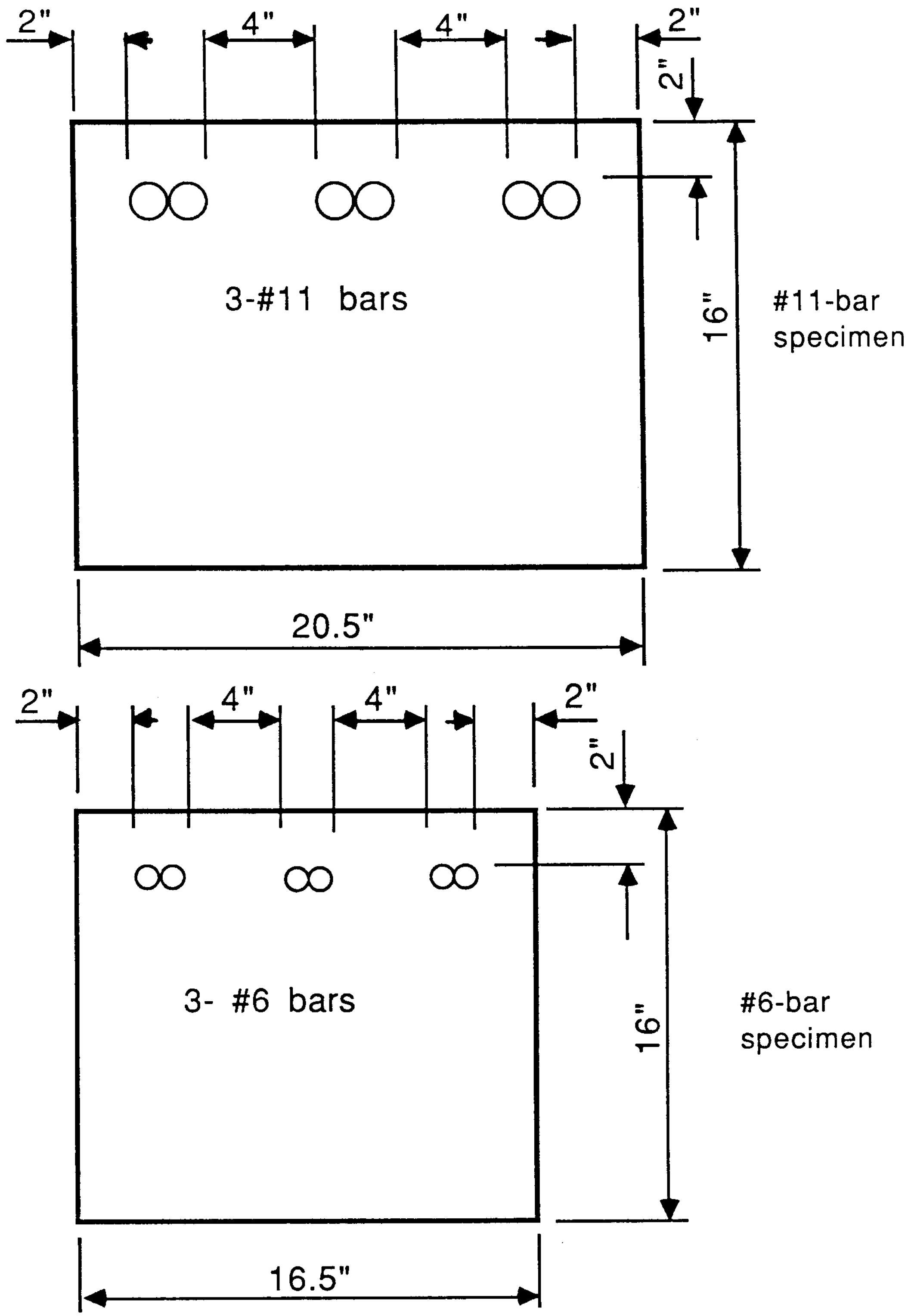


Figure 2.2 Beam Cross Sections

bars. Modes of failure are discussed in Chapter 3. The bars were removed from the tested beams and were used to construct another set of specimens, X-6-4r, with the same variables as X-6-4. The splice length was increased to 24 in. and the top cover was reduced to 3/4 in. The remaining beams with #6 bars were redesigned using a cover of 3/4 in. and a longer splice length. The side cover and overall beam dimensions were not changed.

After the section properties were chosen, the lengths of the test specimens were determined. Since determining the spacing and width of cracks was also an objective, it was desired to have a constant moment region long enough to allow random distribution of cracks outside of the splice. Flexural cracks usually form at or near each end of the splice. Also, cracks tend to form directly over or near the supports. The length of the constant moment region was selected so that the location and spacing of cracks was not influenced by these discontinuities.

In order for the beam to fail at the splice, the remaining portion of the beam was designed to develop yield in the steel. Therefore the length of the bars outside the constant moment region was at least equal to the required development length.

Two practical considerations controlled the final lengths of the specimens. The tie-down anchors in the reaction floor at the Ferguson Structural Engineering Laboratory are spaced four feet in each direction. The length of each specimen had to be a multiple of 4 ft. It was also desired to cast only two sizes of specimens: one for the #6 bar specimens and another for the #11 bar specimens.

A length of 12 ft. between loading points with a 4 ft. constant moment region was chosen for the #6 bar specimens. A length of 20 ft. between loading points with a 9 ft. constant moment region was chosen for the #11 bar specimens. These lengths provided adequate constant moment regions and shear spans long enough to develop yield in the steel. Six inches was added to each end of the specimens to allow area for a loading beam. This resulted in overall lengths of 13 ft. for the #6 bar specimens and 21 ft. for the #11 bar specimens (Fig. 2.3).

2.4 Materials

In an effort to keep companion specimens identical, all the bars of each size were from the same heat of steel. This

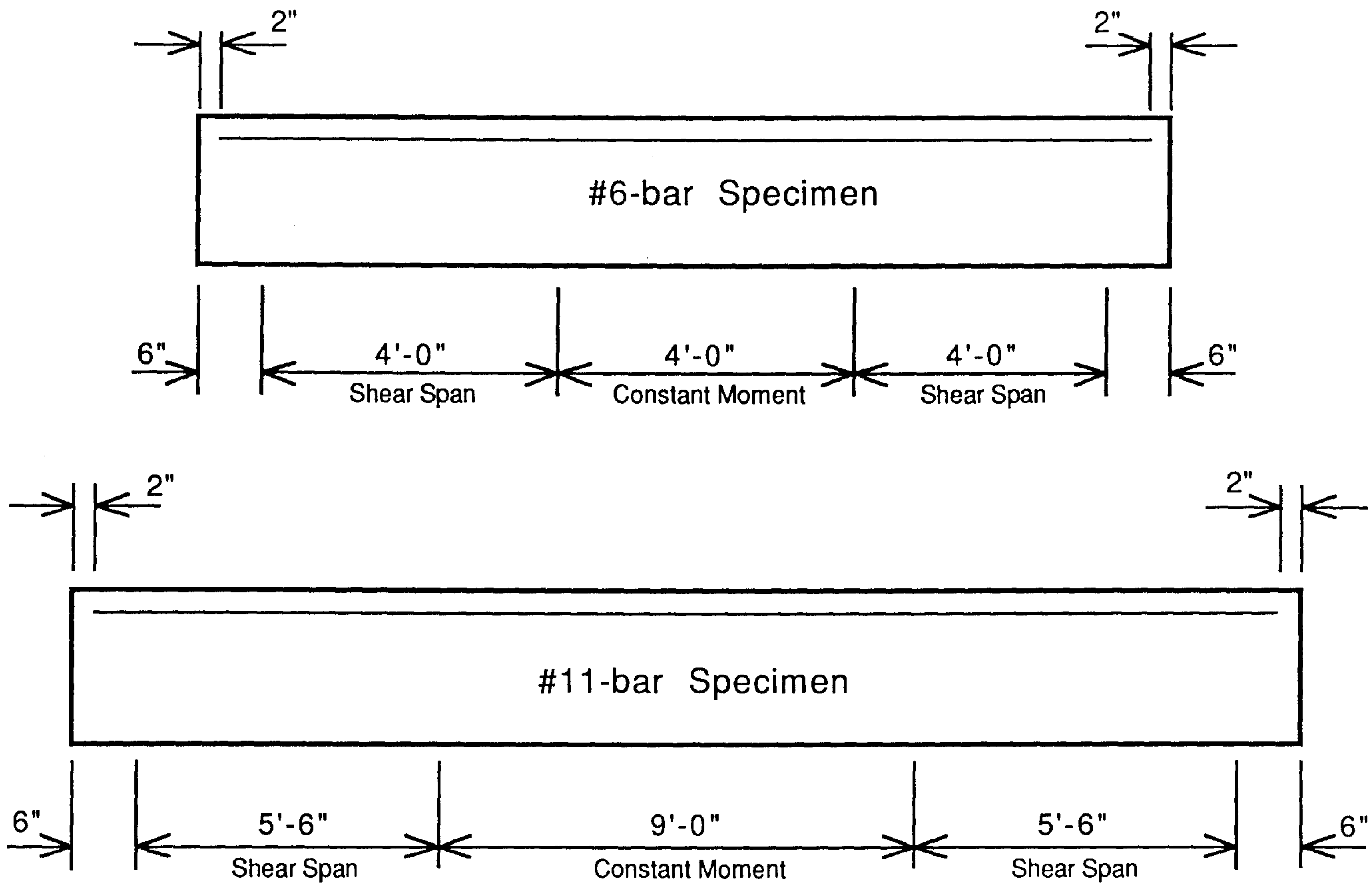


Figure 2.3 Beam Dimensions

insured that coated bars and uncoated bars had identical deformations and mechanical properties. All bars had a diamond deformation pattern and were grade 60. The required number of bars was coated by the steel manufacturer. The coated bars were then individually wrapped and bundled in cardboard to prevent damage to the epoxy coating. Both the coated and uncoated bars were tagged with the lot number.

A mill test report for the heat of steel used was sent with the bars. In addition, 2 bars of each size tested at the laboratory confirmed the values in the mill test report. Both bar sizes exhibited a well-defined yield plateau. Tensile properties from the mill test report and deformation measurements for each bar size are included in Table 2.2.

Prior to casting, the thickness of the epoxy coating was measured. The coating thickness was measured with a Mikrotest Thickness Gage (Figs. 2.4 and 2.5). Each bar sample was measured in three places on each side of the bar, a side being considered the area between longitudinal ribs. The ends of the bar sample, where the thickness was greater, were avoided. Table 2.3 shows the average coating thickness for each coated bar used in the program. The actual measured values varied significantly from the average values shown. The distribution of measured coating values for the bars in each specimen are shown in Fig. 2.6.

Two concrete mixes were required for construction of the specimens. For the first five specimens, a typical 5-sack mix ordered from a ready-mix concrete company was used to obtain the nominal 4000 psi concrete. Another mix design was used for all the high-strength casts. The nominal 8000 psi mixes were designed to reach 12,000 psi at 28 days. With extra water added to the mix, the strength was monitored and the beams were tested when they reached approximately 8000 psi. The mix proportions for the 8000 psi and 12,000 psi nominal strength concrete are outlined below.

Large Aggregate (Dolometic Limestone)	1827#
Small Aggregate (River Sand)	1058#
Cement (Type I)	698#
Fly Ash (Class C)	299#
Water	249#

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

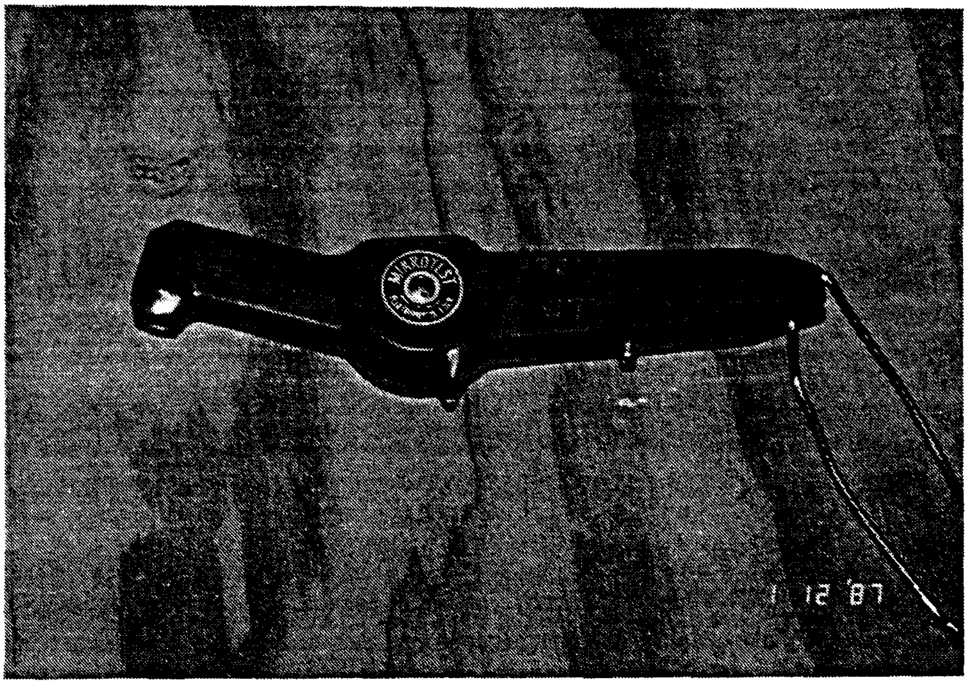


Figure 2.4 Mikrotest Thickness Gage

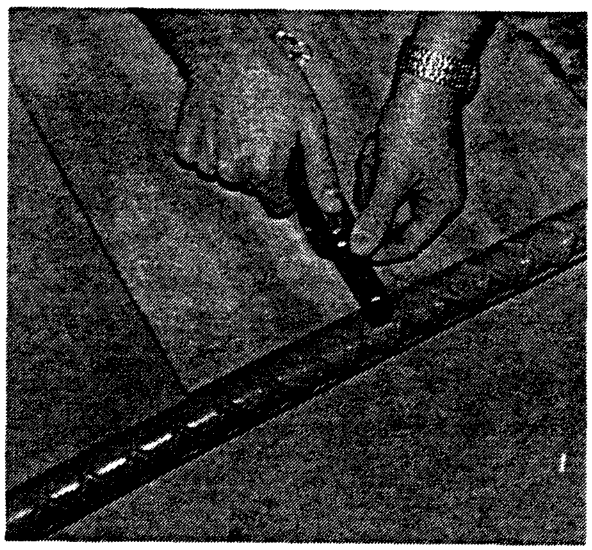


Figure 2.5 Measuring Coating Thickness

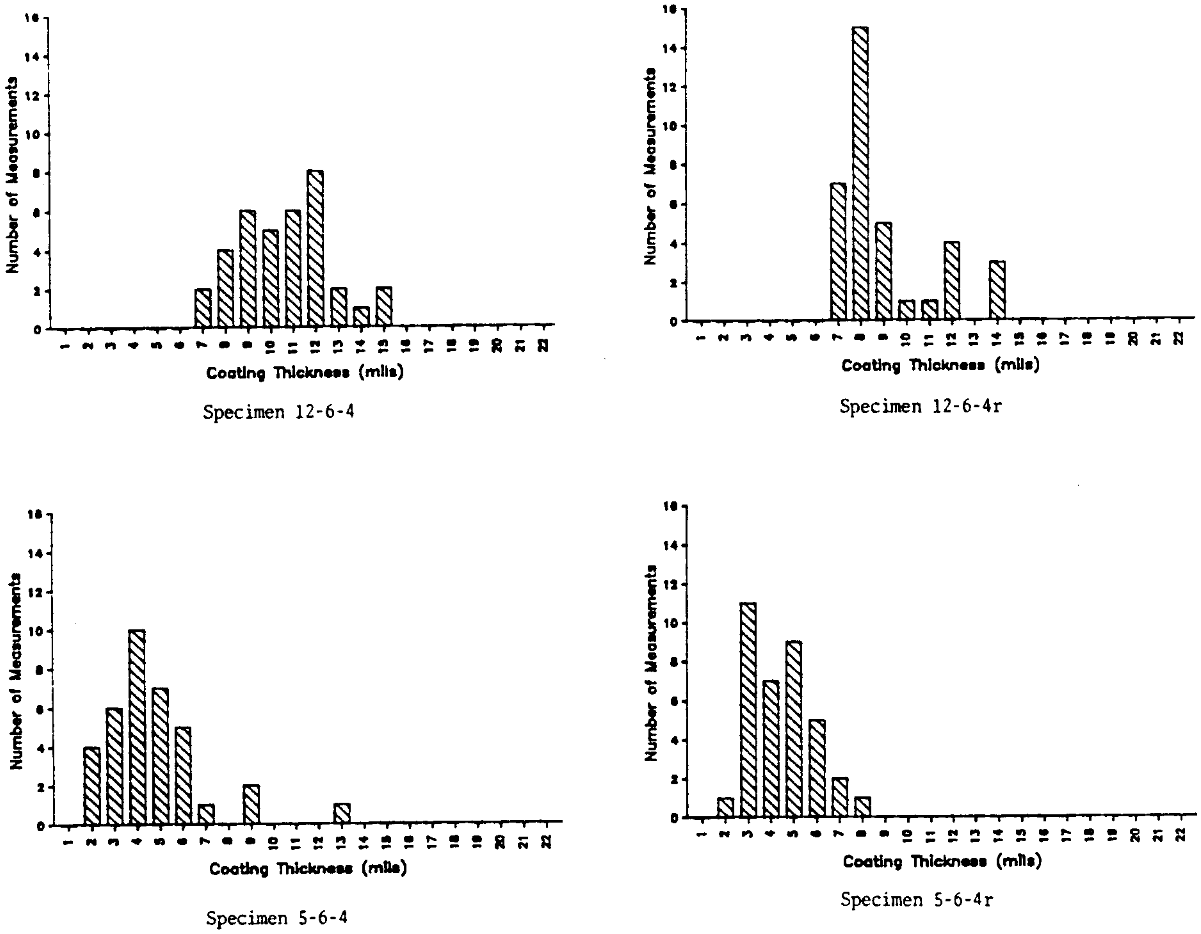
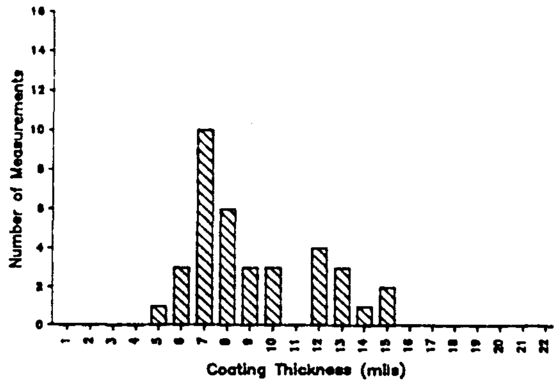
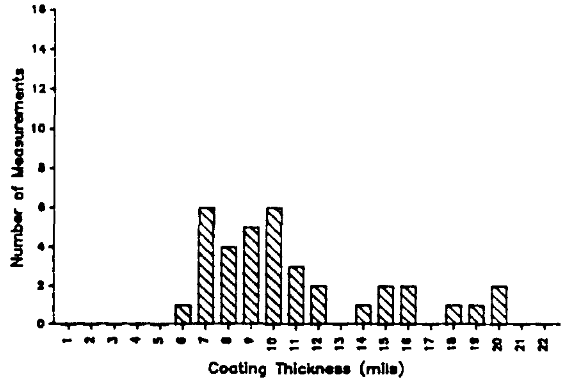


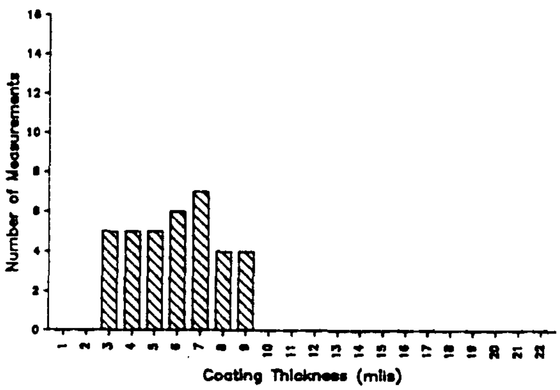
Figure 2.6 Distribution of Coating Thickness Measurements



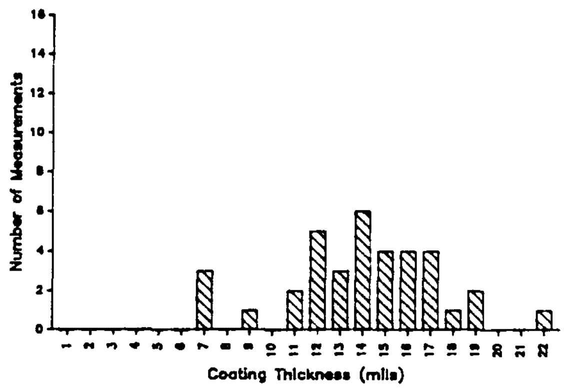
Specimen 12-11-4



Specimen 12-11-4b

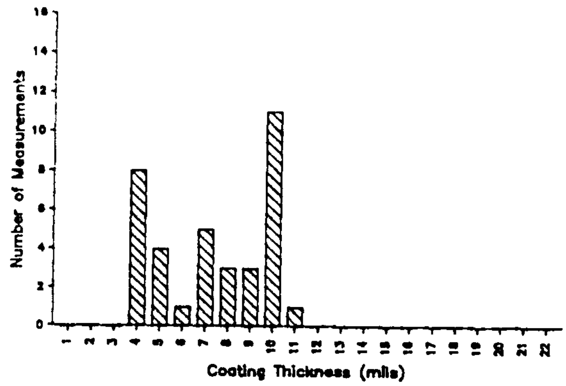


Specimen 5-11-4

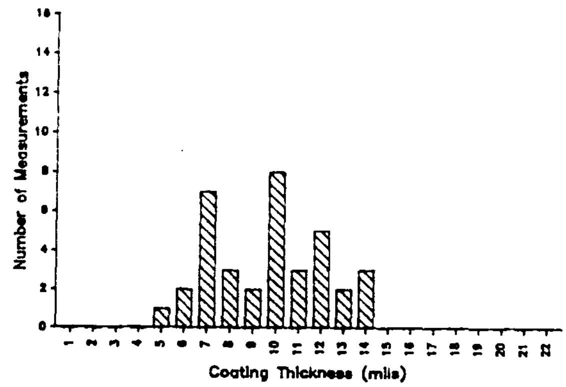


Specimen 12-6-8

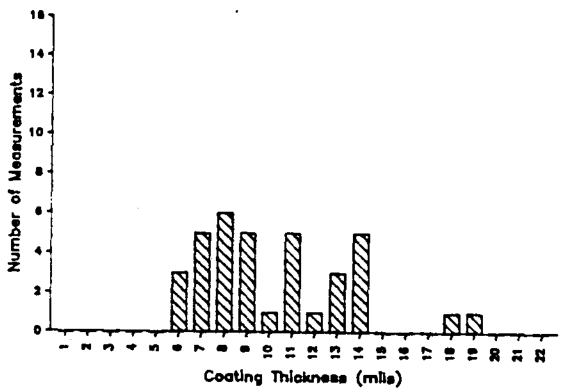
Figure 2.6 Distribution of Coating Thickness Measurements (continued)



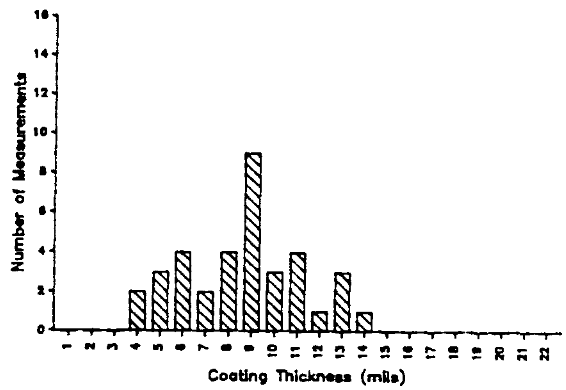
Specimen 12-11-8



Specimen 12-11-12



Specimen 12-6-12



Specimen 12-11-12b

Figure 2.6 Distribution of Coating Thickness Measurements (continued)

2.5 Construction of Specimens

2.5.1 Formwork The formwork was designed so that all beams in a series could be cast simultaneously from the same batch of concrete. A formwork base was constructed which could hold either three #11-bar specimens or three #6-bar specimens. Figure 2.7 shows the formwork. Dividers between beams were attached to the base of the formwork in two different patterns to provide the correct dimensions for either beam size. The dividers were fastened to the base by threaded rods which went through the middle of each divider into tie-down nuts in the base of the formwork. This allowed relatively easy assembly and stripping of the formwork.

2.5.2 Fabrication of cages The steel cages were fabricated on racks and then placed in the formwork. The bars were cut to provide the correct splice length and overall length of the beams.

Coated bars tend to have a thicker coating at the end of the bars because the ends are coated manually. Therefore the sawed ends of the bars were spliced together to give the most uniform coating thickness over the length of the splice. (Sawed ends were produced when cutting bars to the length required for the beams and cutting bars for coupon tests.)

Hoop stirrups were tied to the longitudinal bars over the length of each shear span. Since a splice failure is a splitting phenomenon, any reinforcement surrounding the splice will improve the bond. Stirrups were left out of the splice region to eliminate any influence of transverse steel on bond performance. The longitudinal bars were spaced along the top of each stirrup to maintain the correct spacing of the bars at the splice. Figure 2.8 shows the layout of the steel cages. Two #3 bars were tied in the corners of the hoop stirrups opposite the test bars.

Since stirrups were located only in the shear spans, a method of holding the splices in the correct location was required. After the cages were placed in the formwork, a bar was placed across the top of the formwork at the center of the beams. Each splice was held in the position by a wire from this bar to maintain the correct top cover (Fig. 2.9).

2.5.3 Casting As noted before, all beams in a series of specimens were cast simultaneously. The first four series, X-6-4, X-11-4, X-11-4b, and X-6-4r, were cast indoors. The concrete was

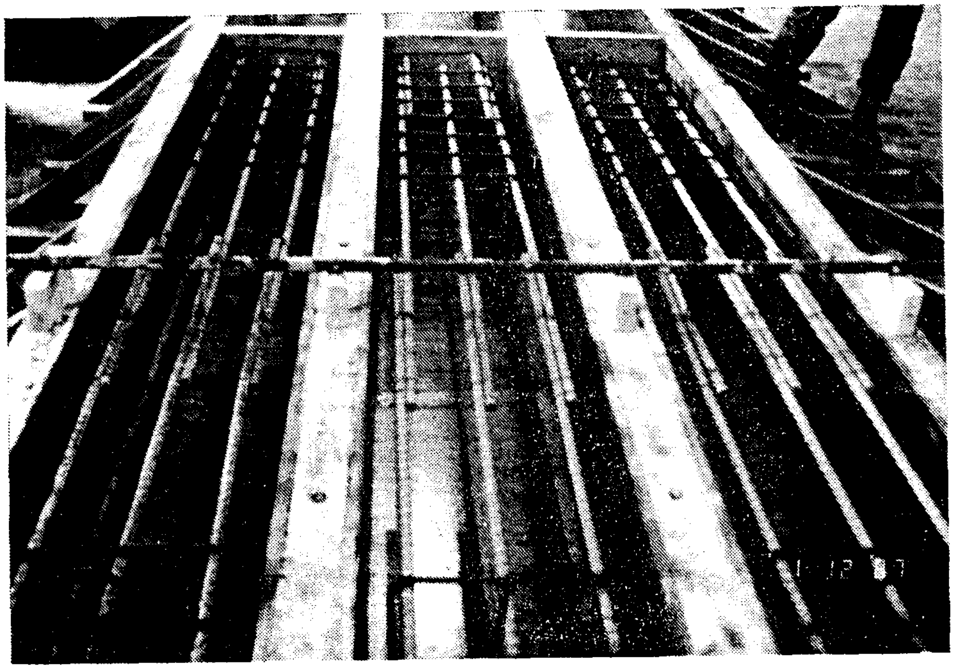


Figure 2.7 Formwork

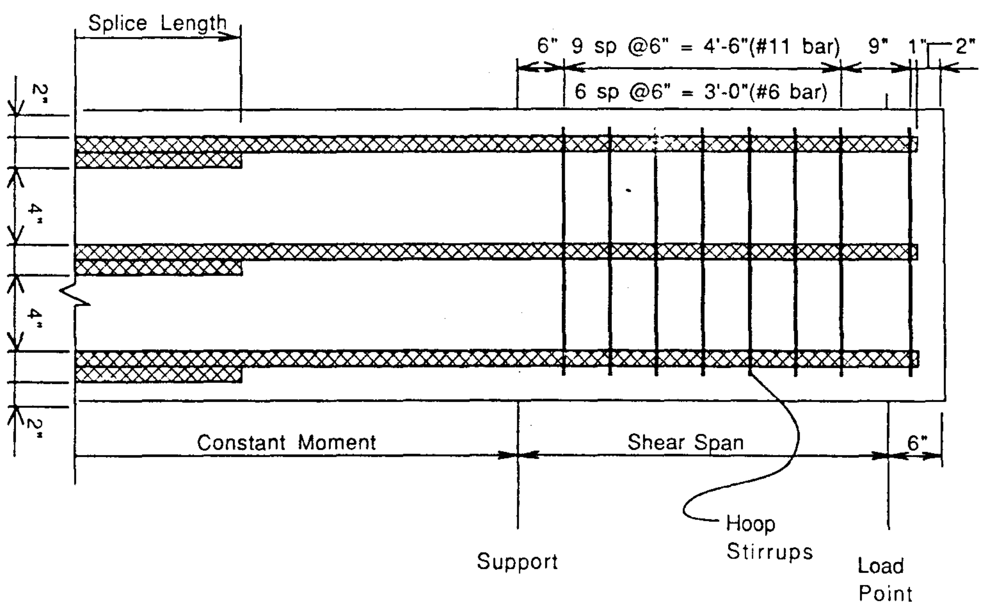


Figure 2.8 Steel Layout

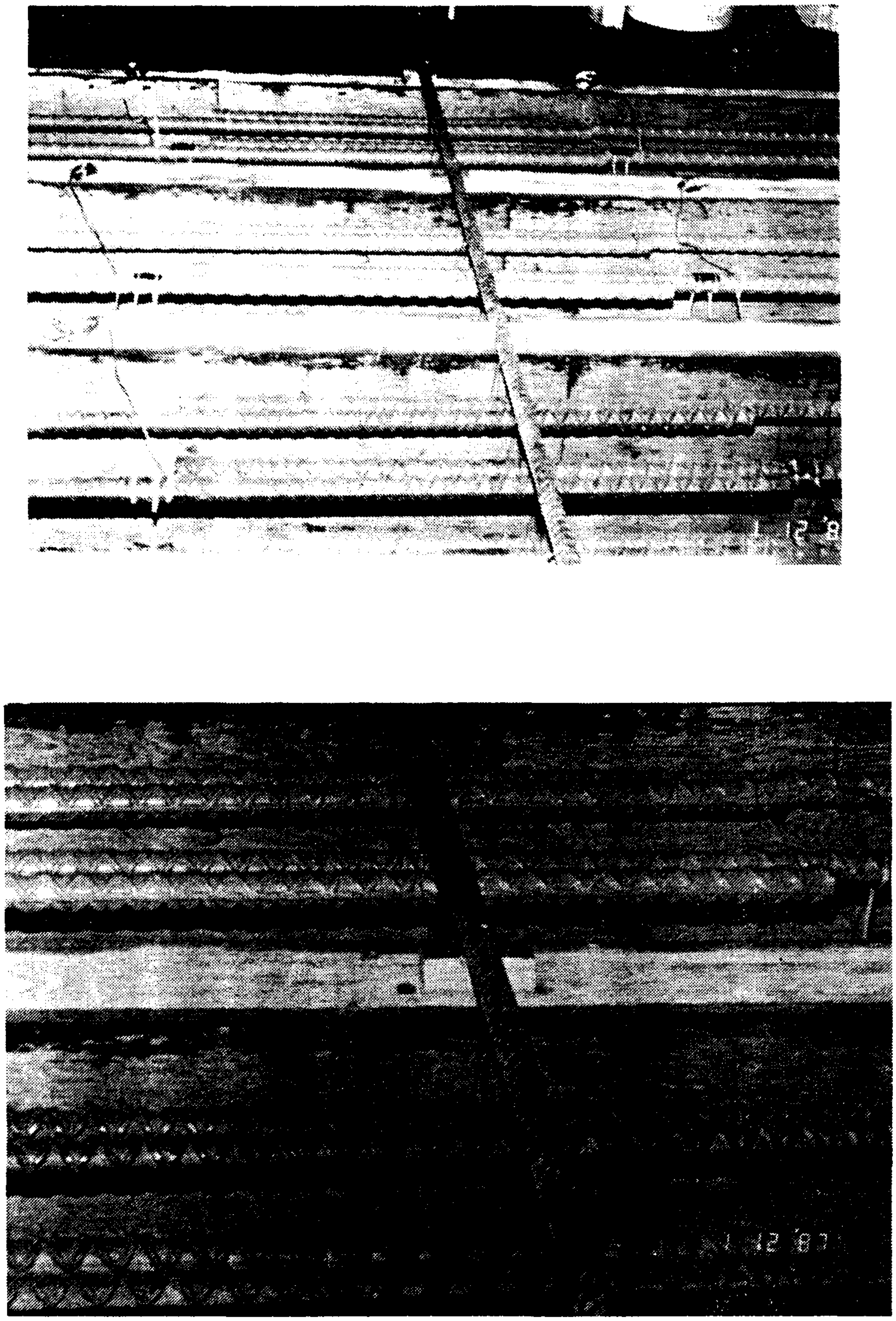


Figure 2.9 Splice Support Bar

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

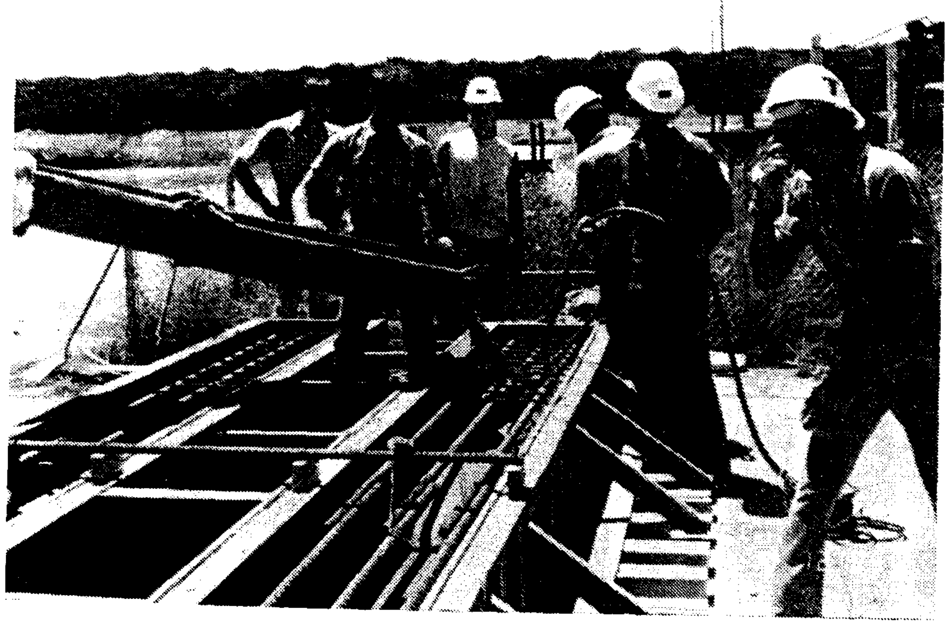


Figure 2.10 Concrete Placement

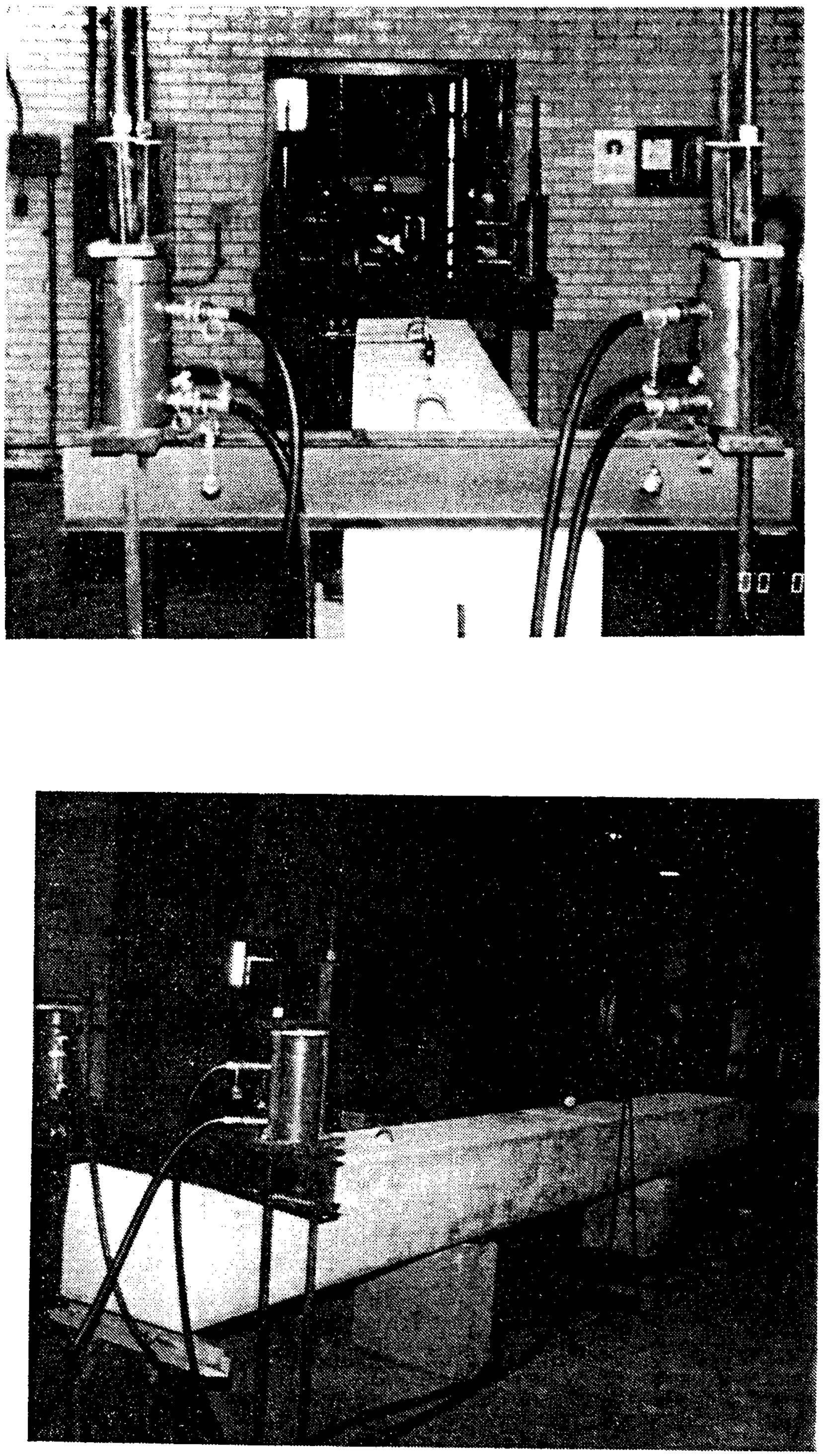


Figure 2.11 Test Setup

support. A steel plate was grouted to the support block and the bottom of the beam at each support to distribute the reaction into the concrete. At one support, the roller was welded to the steel plate simulating a pin connection. At the other support, the bar was free to translate, simulating a roller connection. Load was applied to the specimen with two 30 ton rams at each end (Fig. 2.12). Tie-down rods transferred the reaction from the rams to the reaction floor.

Load was applied in increments of approximately one kip until the beam was cracked along the length of the constant moment region. After the entire constant moment region was cracked, load was applied at increments of approximately two kips. The load was monitored by a 5000 psi pressure transducer which was read with a strain indicator. The pressure transducer and strain indicator were connected to each ram and calibrated in a 60 kip universal testing machine.

At each load stage the maximum load was read. After reaching a desired load, the pressure line was closed. However, the load dropped slightly while reading deflections and crack widths. The highest load at each stage was recorded.

At each load stage, deflection readings were taken and flexural cracks were marked and measured. The crack widths were measured with a crack width comparator. In the first series of tests, each crack was measured in two places. Results showed that the width of the crack varied only slightly along its length. Therefore in subsequent tests, each crack was measured in only one location. Deflection readings were taken with 0.001 in. dial gages at one end (at the point of loading) and at the center of the beam.

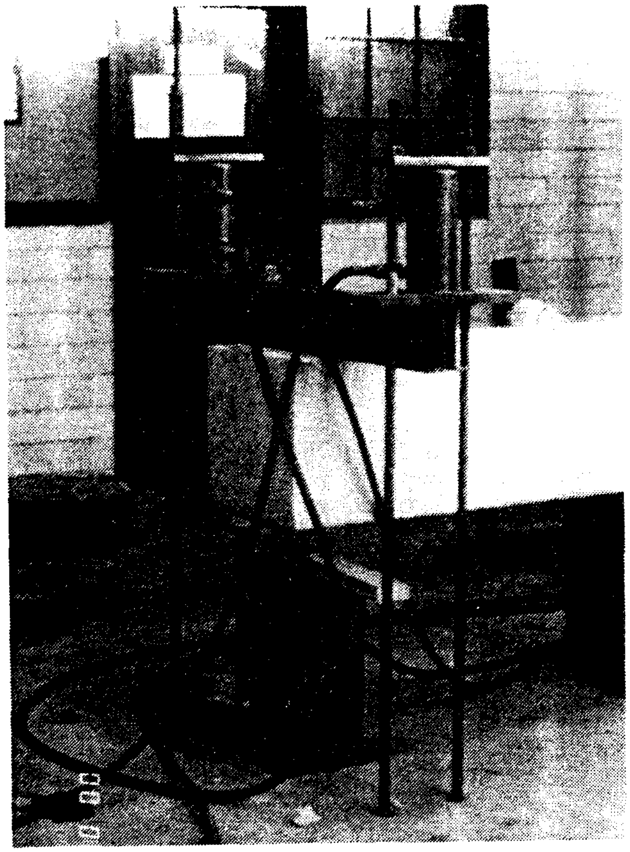


Figure 2.12 Loading System

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.

All the specimens had originally been designed so the top cover would be equal to the side cover at the splice. The clear spacing between bars was twice the side or top cover. By reducing the top cover to 3/4 in. on the #6 bars, the pattern of the splitting failure was changed.

3.2.2 Longitudinal Cracking In the #11 bar specimens longitudinal cracks formed in the top cover directly over the spliced bars and in the side cover adjacent to the bars. The final mode of failure was a face-and-side split failure (Fig. 3.1).

In the #6 bar specimens, longitudinal cracks formed in the top cover directly over the spliced bars, but did not form in the side cover, The final splitting pattern was a V-notch failure (Fig. 3.2).

In the uncoated bar specimens longitudinal cracks began forming at about one-half the maximum load. They were primarily visible over the two exterior splices. The interior splice seldom showed signs of distress until failure. Longitudinal cracks began forming in the coated bar specimens at slightly lower loads than in the uncoated bar specimens. The crack patterns were very similar. However, longitudinal cracks in the coated bar specimens were followed by a splitting failure with little increase in the load. Longitudinal cracks in the uncoated bar specimens were maintained even though the load increased significantly up to failure.

3.2.3 Appearance After Failure After a splitting failure occurred in the tests, the top cover over the splice was removed to reveal the plane of failure across the splice. There was no evidence of adhesion between the epoxy-coated bars and surrounding concrete. The concrete in contact with the epoxy-coated bars had a smooth glassy surface as if a bond-breaker had been applied (Fig. 3.3). The patterns left in the concrete by the deformations of the bars were in perfect condition. There were no signs of the concrete being crushed against the bar deformations. The epoxy-coated bars in the splice were very clean with no concrete residue left on the deformations or the shaft of the bar (Fig. 3.4).

The uncoated bars, however, showed evidence of good adhesion with the concrete. Concrete particles were left firmly attached to the shaft of the bar, with large deposits left on the sides of the deformations (Fig. 3.5). The concrete cover which was in contact with the bars was very dull and rough. Pieces of

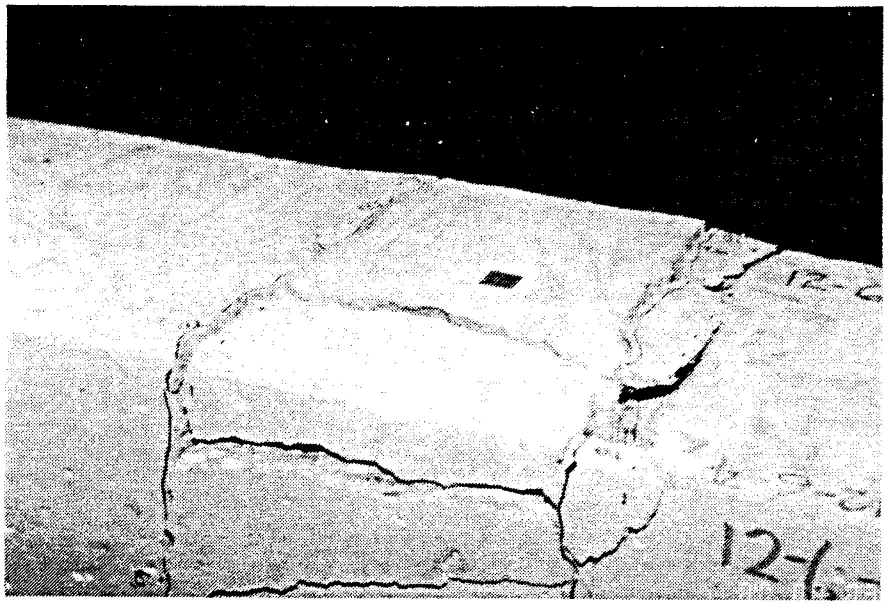


Figure 3.1 Face-and-Side Split Failure

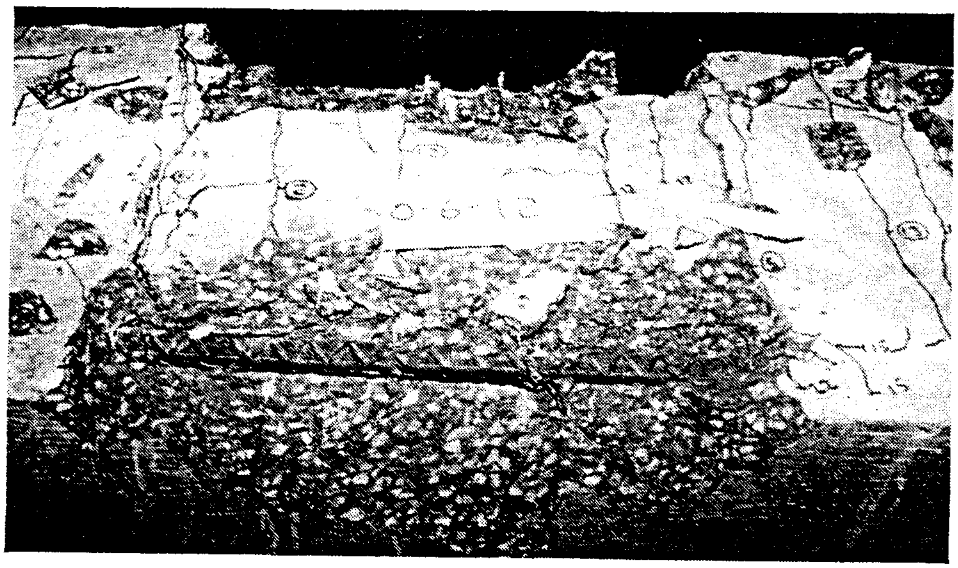


Figure 3.2 V-Notch Failure

Figure 3.3 Cover on Coated Bars After Test

Figure 3.4 Coated Bars After Test

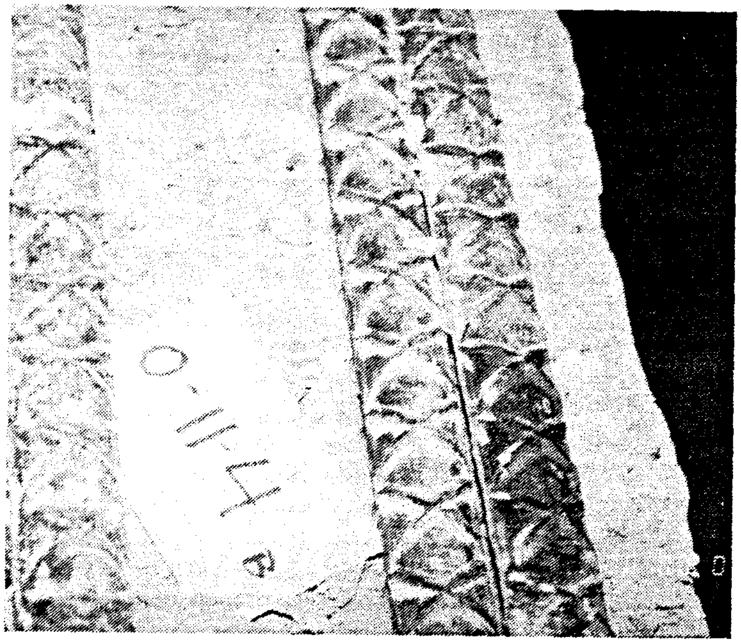


Figure 3.5 Uncoated Bars After Test

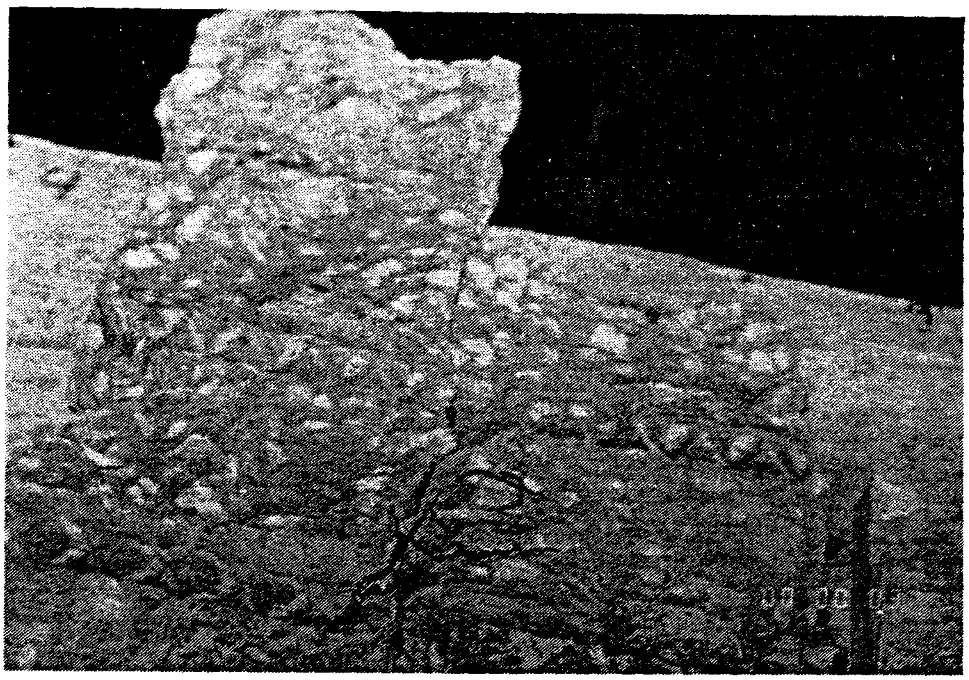


Figure 3.6 Cover on Uncoated Bars After Test

mill scale had been pulled off the bars and were still in contact with the concrete. The patterns in the concrete left by the deformations showed signs of crushing due to bearing against the deformations (Fig. 3.6).

3.3 Results of Beam Tests

The parameters of each specimen and the data measured during the tests are shown in Table 3.1. As can be seen from the test data, the coated bars developed significantly lower stresses than the uncoated bars. Since the cover varied slightly between specimens, a direct comparison using this data would not be complete. In the following section, the bond strengths are compared accounting for variations between specimens. The performance of epoxy-coated bars is also evaluated in terms of stiffness, and crack width, and crack spacing in subsequent sections. As this set of tests included several variables, the performance of coated bars versus uncoated bars was evaluated for the range of each variable.

3.4 Bond Strength

In each test, the mode of failure was a splitting failure at the splice region. Therefore, the splice reached its capacity and the bond strength could be determined directly from the stress developed in the steel. The bond strength was based on an average stress along the length of the splice. It was calculated by dividing the total force developed in the bar by the surface area of the bar over the splice length, $u = f_s\, d_b/4\, l_s$.

The steel stress developed by each specimen was determined by analyzing the section based on cracked, elastic behavior, ignoring the tensile stresses in the concrete below the neutral axis. Each specimen was first analyzed using the Hognestad stress block, which considers the non-linearity of the concrete stress-strain diagram. Each specimen was also analyzed assuming a linear stress-strain diagram. Comparison of steel stress values obtained from each analysis showed less than 3% difference. For simplicity, the linear assumption was used as the basis for determining the steel stresses.

In three series of specimens, X-6-4r, X-6-8, and X-6-12, the uncoated bars began yielding before a splitting failure was reached. This can be seen in the load deflection curves of these

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	5/3	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

specimens, Figs. 3.13, 3.16, and 3.18. The load at which yielding occurred in these specimens is clearly defined in the load deflection curves. Based on the load at yield, the steel stresses were calculated using the linear model. For specimen 0-6-4r, the calculated steel stress was 62.4 ksi compared to the measured yield strength of 63.3 ksi. In specimens 0-6-8 and 0-6-12, which were high-strength concrete, the resulting steel stresses were higher than the measured yield strength. This difference may be due to the use of the linear model for high-strength concrete and the assumption that the concrete below the neutral axis did not contribute to the flexural capacity. Since the actual yield strength of the steel was known, the stress developed in the specimens which yielded was taken as the actual yield strength, 63.3 ksi.

In each series of specimens, the concrete strength; bar size and characteristics; casting position; and section properties were held constant. However, when the cover was reduced to 3/4 in. on the #6 bars, it was difficult to maintain equal cover between two beams in a series. Although the cover varied a maximum of 1/4 in. within a series, this could have a significant effect on the results of the tests due to the small nominal cover.

The bond strengths were normalized with respect to cover, bar diameter, splice length, and concrete strength to account for the difference between beams and to compare the results between series of tests.

The bond strength equation used as the basis for the current splice and development length provisions in ACI 318 [1], $u = 9.5\sqrt{f'_c}/d_b$, does not consider the depth of cover or the length of the splice. Cover is not considered to influence bond strength and the bond stress is assumed constant over the length of the splice. Equation 3.2, described in Chapter 1, was used to normalize the results of the bond tests. It considers the depth of cover, concrete strength, bar diameter, and splice length in computing the bond strength. For bars with no transverse reinforcement providing confinement:

$$u/\sqrt{f'_c} = [1.2 + 3(c/d_b) + 50(d_b/l_s)] \quad \text{Eq. 3.2}$$

As indicated by the third term in the brackets, as the splice length increases, the normalized bond stress per unit length decreases. The bond stress, in reality, is not distributed uniformly along the length of the splice. The bond stress is higher at the loaded end of each spliced bar. Equation

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

Table 3.2 Comparison of results

Specimen	Measured Bond Strength (psi) utest	Computed Bond Strength (psi) u-ACI*	Computed Bond Strength (psi) u-theor	Bond Efficiency (psi) u-ACI*	Bond Efficiency (psi) u-theor	Bond Ratio coated/ uncoated
12-6-4	516 520	695 ✓690	803 800	0.74 .83	0.64	0.62
5-6-4	722 720	695	803	1.04 1.15	0.90	0.87
0-6-4	830 830	695	803	1.19 1.33	1.03	1.00
12-6-4r	350 350	493 660	389 390	0.74 .56	0.90	~~0.76~~ .71
5-6-4r	374 370	493	358 360	0.76 .59	1.05	~~0.88~~ .76
0-6-4r	495 Y500	493	420 420	1.00 .80	1.18	1.00
12-11-4	277 280	400 450	526 530	0.69 .70	.53	.65
5-11-4	298 300	400	526	0.75 .75	.57	.70
0-11-4	424 420	400	526	1.06 1.05	.81	1.00
12-11-4b	244 240	372 370	486 490	0.66 .65	.50	.54
0-11-4b	449 450	372	486	1.21 1.22	.93	1.00
12-6-8	410 410	739 ✓960	587 590	0.55 .66	.70	~~.60~~ .55
0-6-8	742 Y 740	739	632 630	1.00 1.18	1.17	1.00
12-11-8	495 500	495 520	901 900	0.96 .96	.55	.61
0-11-8	789 790	516	877 880	1.53 1.52	.90	1.00
12-6-12	482 480	739 1200 (625)	678 680	0.65 .77	.71	~~.70~~ .65
0-6-12	742 Y740	739	735 740	1.00 1.18	1.01	1.00
12-11-12	662 660	581 ✓580	961 960	1.14 1.14	.69	.72
0-11-12	918 920	581	961	1.58 1.58	.96	1.00
12-11-12b	539 540	555 ✓560	918 920	0.99 0.99	.59	.64
0-11-12b	842 840	555	918	1.52 1.52	.92	1.00
Avg. of all - coated bars:				~~0.80~~ .81	0.69	~~0.66~~ 0.67
S.D.:				~~0.17~~ .19	0.17	~~0.07~~ 0.09

*Upper limit on bond stress is 625 psi

Uncoated 1.26 0.99
0.24 0.12

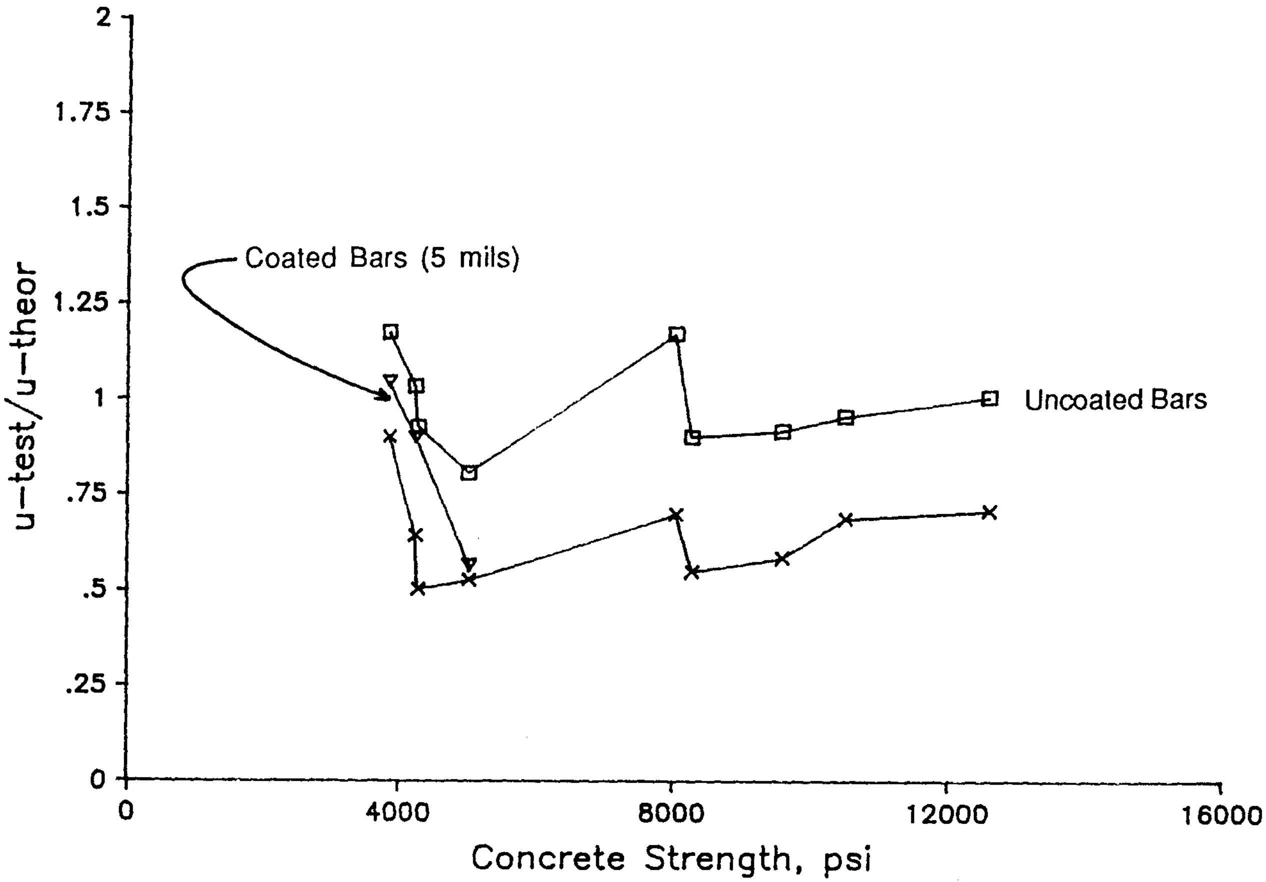


Figure 3.7 Bond Efficiency vs. Concrete Strength

which connect the points, the coated bar specimens follow the same pattern as the uncoated bar specimens. Random errors in construction may have caused slight variations in the bond efficiency, however, the variations were consistent between specimens within any series.

To indicate the relationship of bond of epoxy-coated reinforcing bars with concrete strength more clearly, Fig. 3.8 shows the bond ratio as a function of concrete strength. The bond ratio is the bond efficiency of the coated bar divided by the bond efficiency of the uncoated bar in the same series. The bond ratio for the bars with nominal coating thickness of 12 mils is between 0.5 and 0.77 for all concrete strengths. It appears that the reduction in bond due to the epoxy coating is not related to the strength of the concrete.

The primary objective in using high-strength concrete was to determine what effect it had on the bond strength of epoxy-coated bars. The results of the tests with high-strength concrete also provide important additional data for understanding bond of reinforcing bars in general. Data on bond strength using high-strength concrete is very limited. Two tests by Chinn, Ferguson, and Thompson [24] had concrete strengths of 7480 psi. A total of 9 specimens tested by Tepfers [25] had concrete strengths ranging from 6000 psi to 13,300 psi. The results of these specimens were included by Orangun, Jirsa, and Breen [4] in the development of Eq. 3.2. Results of the tests with high-strength concrete are shown in Table 3.3. In the current tests, 5 series of specimens had concrete strengths greater than 8000 psi.

Equation 3.2 is very accurate in evaluating the bond strength for the current and previous tests done on bond with high-strength concrete. Figure 3.9 shows the bond efficiency (using Eq. 3.1 with the square root of the concrete strength) for all the available tests with high-strength concrete plotted versus the concrete strength. For concrete strengths from 6000 psi to 14,000 psi, the average bond efficiency is 1.06 with a standard deviation of 0.19. Although the average bond efficiency is greater than 1, a large number of points are less than 1.

It has been suggested [23] that $4(f'_c)^{1/3}$ may be a better indicator of the bond strength than $\sqrt{f'_c}$ when using high-strength concrete. With $4(f'_c)^{1/3}$ is used in Eq 3.2, replacing $\sqrt{f'_c}$ in the computation for bond efficiency, a second set of points plotted in Fig. 3.9 is obtained. Up to a concrete strength of 6000 psi, the bond strength predicted by Eq. 3.2 using $4(f'_c)^{1/3}$ and $\sqrt{f'_c}$ is

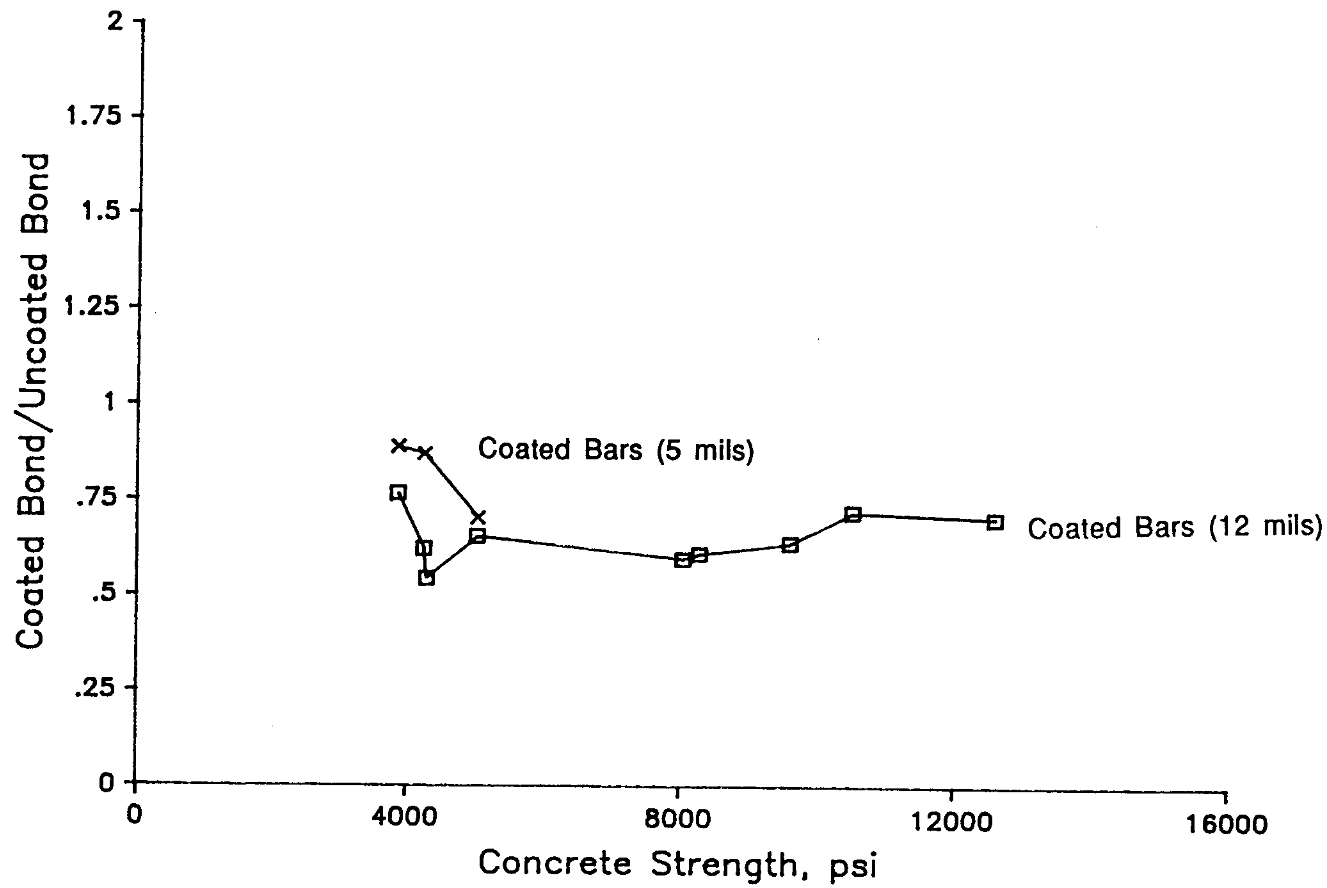


Figure 3.8 Bond Ratio vs. Concrete Strength

Table 3.3 Previous tests using high-strength concrete

Researcher	l_s (in.)	d_b (in.)	c_b (in.)	c_s (in.)	f'_c (psi)	u-test (psi)
Chinn et al. [24]	11	.75	1.39	1.1	7480	737
	16	.75	1.56	1.1	7480	600
Tepfers [25]	52	16	1.9	2.45	13300	643
	52	16	1.8	2.45	12540	490
	52	16	2.1	2.43	6570	660
	52	16	1.6	2.45	8120	749
	52	16	1.7	2.45	9095	677
	52	16	2.1	2.475	6620	539
	52	16	2.3	2.42	6200	714
	52	16	2.6	2.40	6270	719
	52	16	1.8	2.52	7490	527

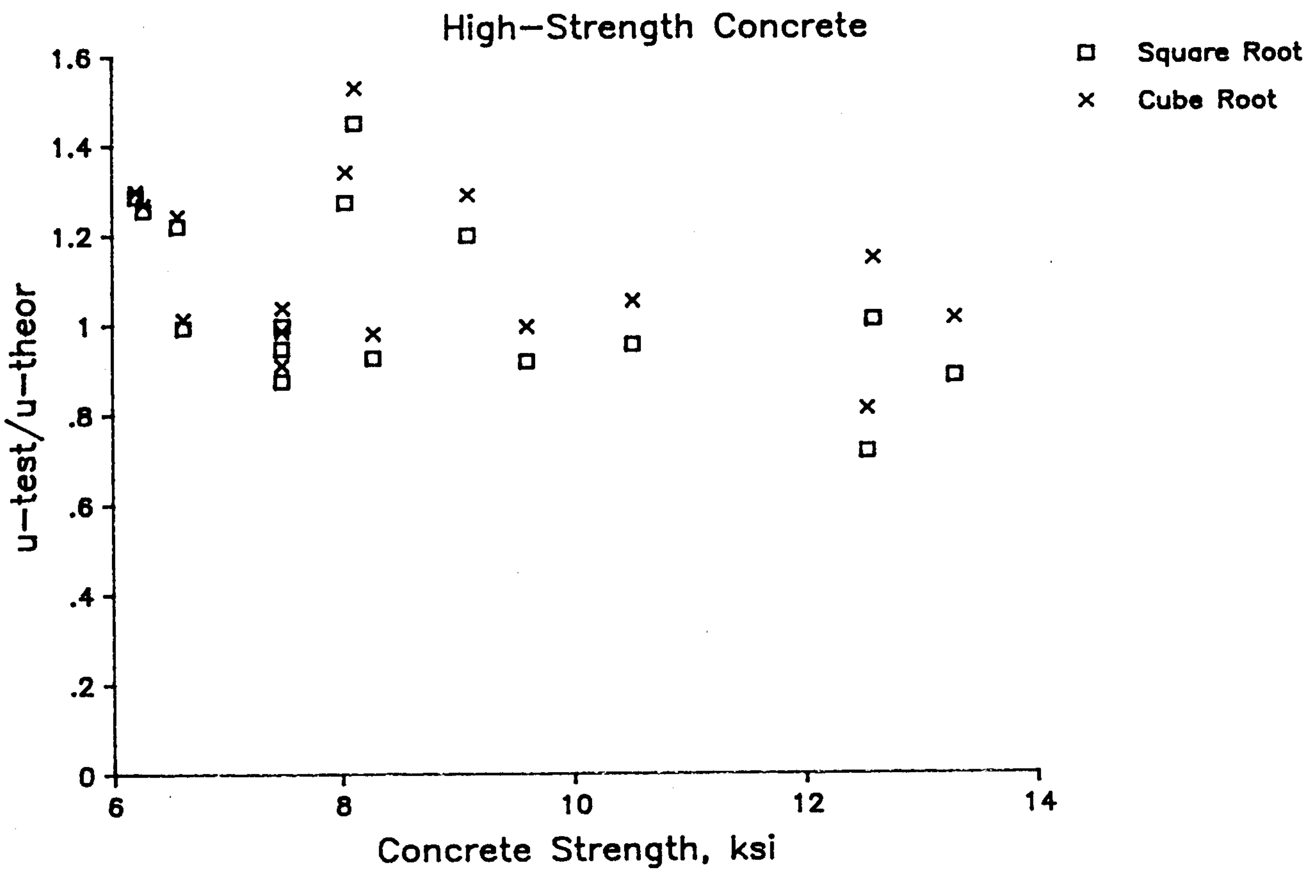


Figure 3.9 Bond Efficiency vs. Concrete Strength

approximately equal. However, as the concrete strength increases, the square root of f'_c predicts a slightly higher value than the cube root, resulting in a lower bond efficiency. When the cube root of f'_c is used, the average bond efficiency is 1.12 with a standard deviation of 0.18. This indicates that only a few specimens have a bond efficiency below 1. Although the amount of data is small, the results indicate that the square root of f'_c may overestimate the bond strength as the concrete strength increases. However, for the range of concrete strengths presently achievable in the field, Eq 3.2 using $\sqrt{f'_c}$ adequately predicts the bond strength.

3.4.2 Bar Size The reduction in bond due to epoxy coating was not influenced by bar size. The average bond ratio for bars with nominal 12 mil coating thickness was 0.67 for the #6 bars and 0.64 for the #11 bars. The uncoated bars in three of the four #6 bar series yielded so the bond ratio reported in these series would have been slightly lower had the uncoated bars not yielded.

3.4.3 Casting Position Although bars in two series of specimens were bottom cast, the effect of casting position on the bond strength of epoxy-coated bars could not be determined because the concrete had a low slump. In each cast, the slump of the concrete was less than 4 in. The quality of bond in top-cast series was not significantly less than the bond in corresponding bottom-cast series. Series X-11-4 and X-11-12, which were top-cast, were repeated with series X-11-4b and X-11-12b, respectively, using bottom-cast bars. The bond strength of the uncoated bars was not affected by the casting position. The bond efficiency of the top-cast specimen, 0-11-4 was 13% lower than the bond efficiency of the corresponding bottom-cast specimen, 0-11-4b. This corresponds to a 15% increase in the development or splice length which is much less than the 40% increase required by ACI 318 for top-cast bars. The bond efficiency of the top-cast specimen, 0-11-12 was slightly higher than the bond efficiency of the corresponding bottom-cast specimen, 0-11-12b.

Recommendations made by Jirsa and Breen [9] on the effect of casting position on bond showed that for low-slump concrete, the ACI-AASHTO casting position factor is very conservative. The concrete used to cast series X-11-4 had a slump of 3-1/2 in. with approximately 12-1/2 in. of concrete cast below the bars. The casting position factor recommended by Breen and Jirsa for this condition is 1.06, indicating that very little loss of bond due to casting position should occur in this series of specimens.

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 Coating Thickness 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

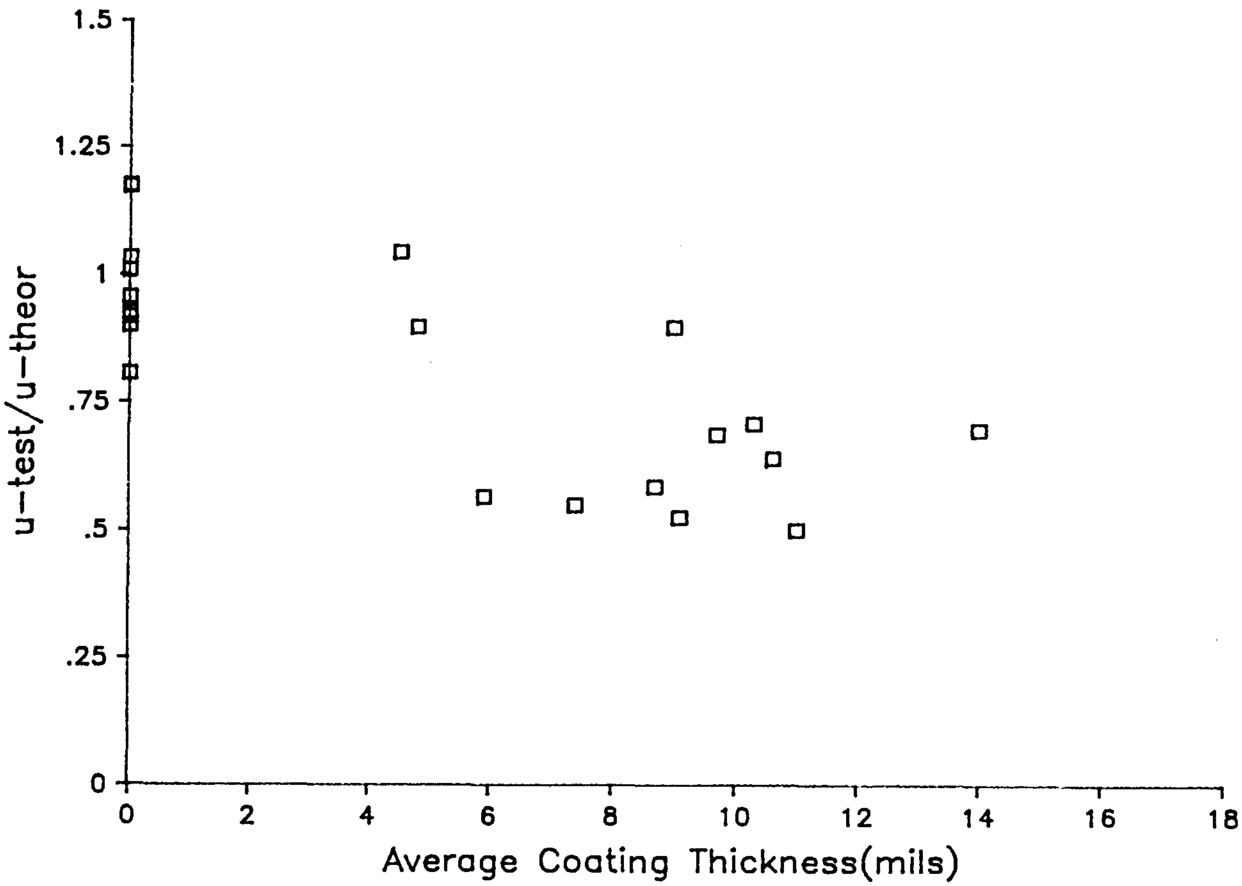


Figure 3.10 Bond Efficiency vs. Coating Thickness

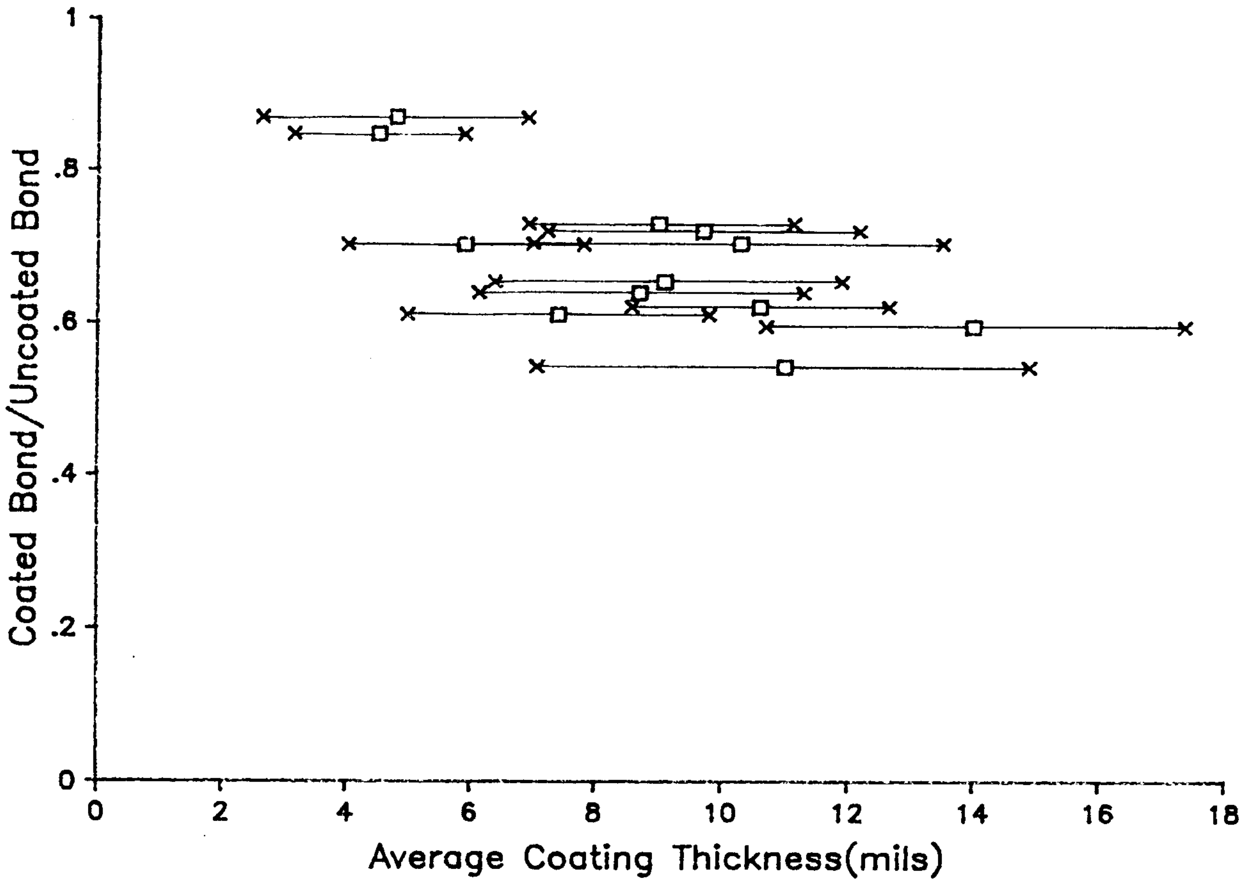


Figure 3.11 Bond Ratio vs. Coating Thickness

coating thicknesses have significantly higher bond ratios than the other coated bar specimens. If these two specimens, which have an average coating thickness less than 5 mils, were excluded from Fig. 3.11, virtually no variation in the bond reduction with coating thickness could be detected. Exclusion of these specimens can be justified since ASTM requires a minimum average coating thickness of 5 mils to prevent corrosion. The average coating thickness of one specimen: 14 mils, exceeded the maximum average coating thickness of 12 mils allowed by ASTM. It could be argued that this specimen should be excluded also, however, Fig. 3.11 shows that this point does not contradict, but rather supports the general trend of the specimens with coating thicknesses between the limits of 5 and 12 mils.

The coating thickness limits of 5 and 12 mils set by ASTM are based on the results of the study conducted at the National Bureau of Standards [5]. The majority of the epoxy-coated bars tested in this study had a nominal coating thickness ranging from 1 to 11 mils. Bars with coating thicknesses between these limits were concluded to have acceptable bond strength. Two bars with a coating thickness of 25 mils exhibited poor bond behavior and were judged unacceptable. Based on this, 12 mils was chosen as a practical upper limit to the coating thickness.

There is no evidence to indicate that the reduction is significantly greater for a coating thickness more than 12 mils. The specimens with a coating thickness of 25 mils tested at the National Bureau of Standards exhibited a greater reduction bond, however, since no tests were conducted on bars with coating thicknesses between 11 and 25 mils, it is unknown at what coating thickness a large decrease in bond results. Results of the current study indicate that the bond reduction remains fairly constant up to an average coating thickness of 14 mils.

Based on the specimens with an average bar coating thickness greater than 5 mils, there appears to be little variation in the bond ratio with coating thickness. The reduction in bond appears to be less in the specimens with an average coating thickness less than 5.0 mils. A coating thickness under 5 mils is not permitted by ASTM standards [7] since a minimum coating thickness must be maintained to prevent corrosion. It is likely that a small coating thickness would be very hard to maintain consistently during the coating process.

3.4.5 Concluding Remarks on Bond Strength There was virtually no variation in bond except between coated and uncoated bars. Epoxy-coated bars with average coating thicknesses above 5 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

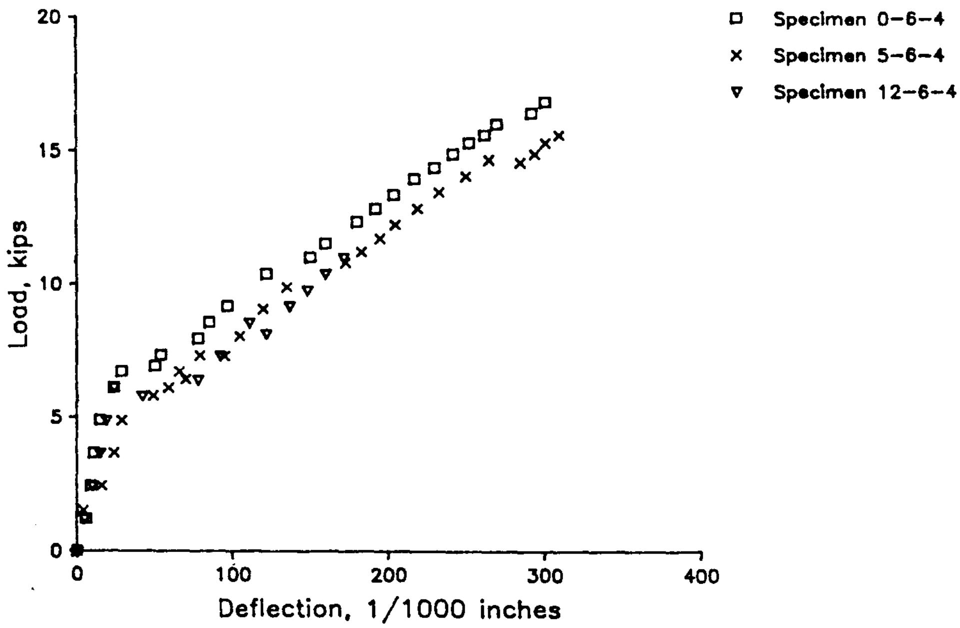


Figure 3.12 Beam End Deflection, Series X-6-4

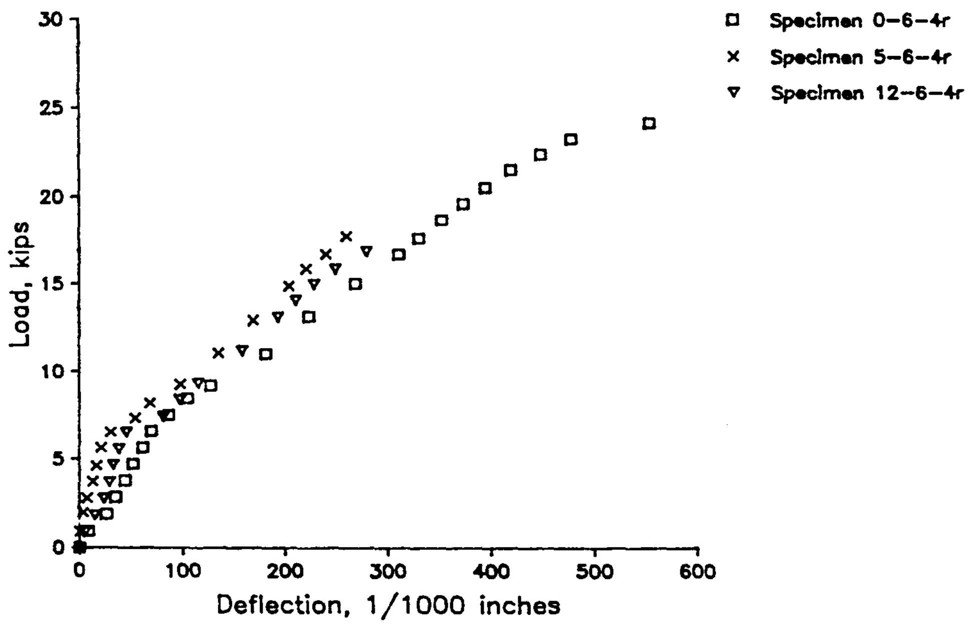


Figure 3.13 Beam End Deflection, Series X-6-4r

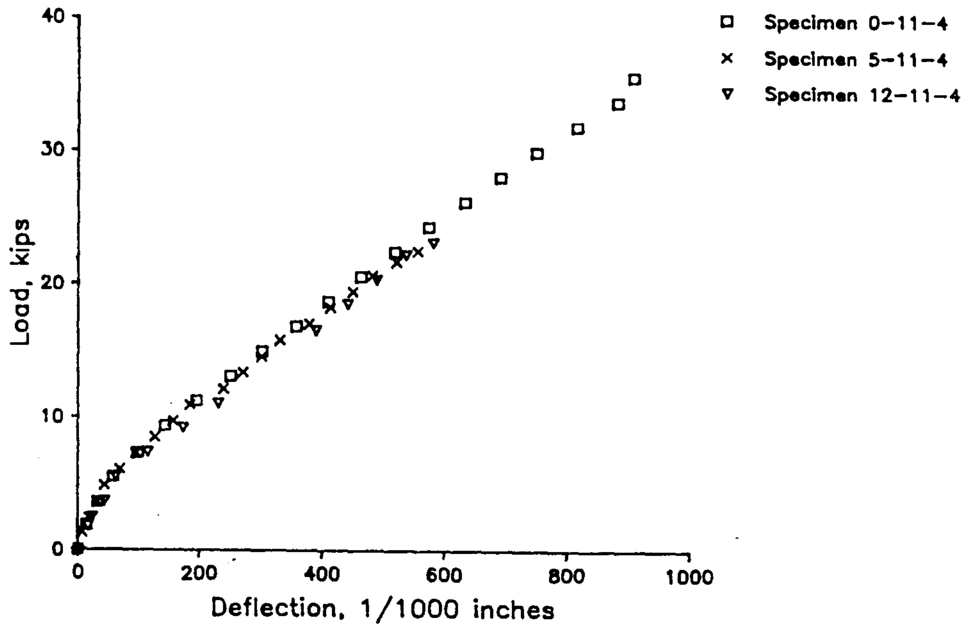


Figure 3.14 Beam End Deflection, Series X-11-4

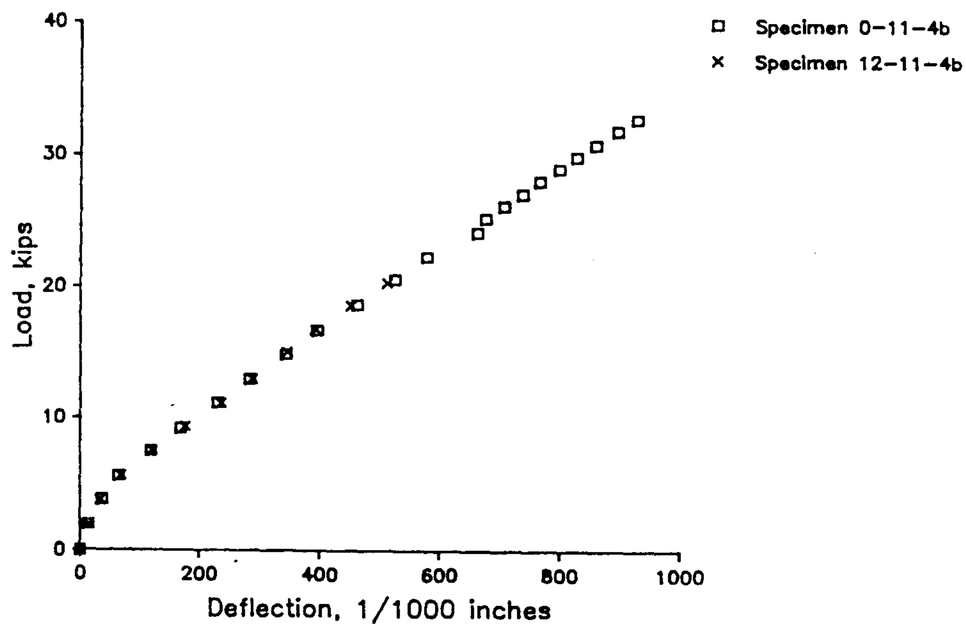


Figure 3.15 Beam End Deflection, Series X-11-4b

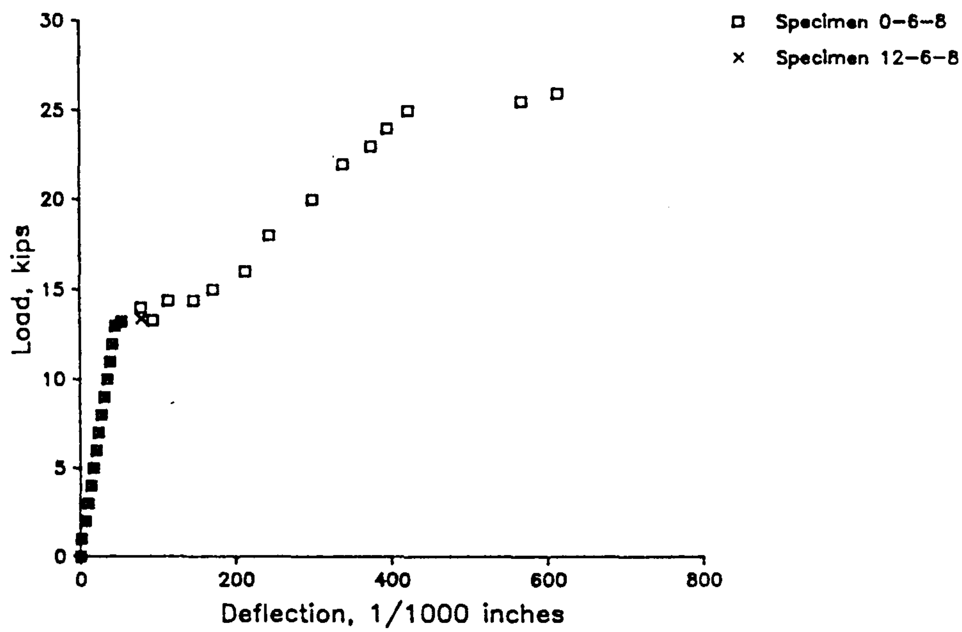


Figure 3.16 Beam End Deflection, Series X-6-8

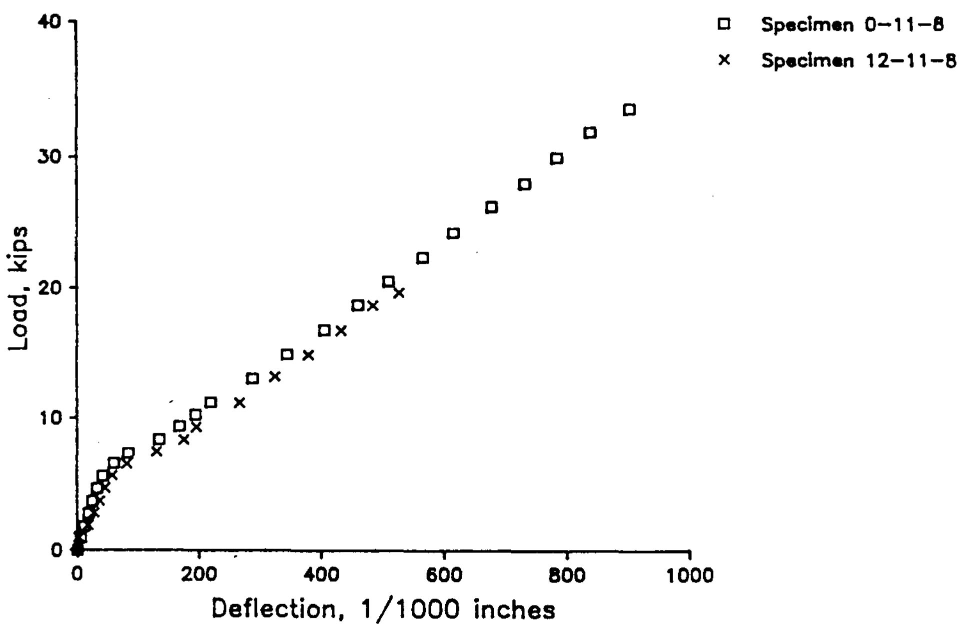


Figure 3.17 Beam End Deflection, Series X-11-8

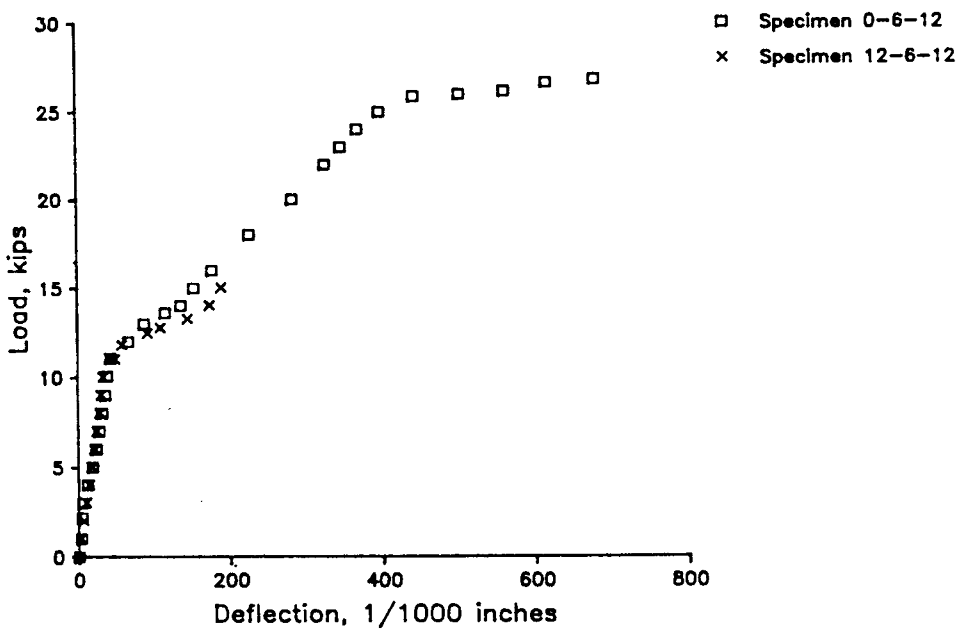


Figure 3.18 Beam End Deflection, Series X-6-12

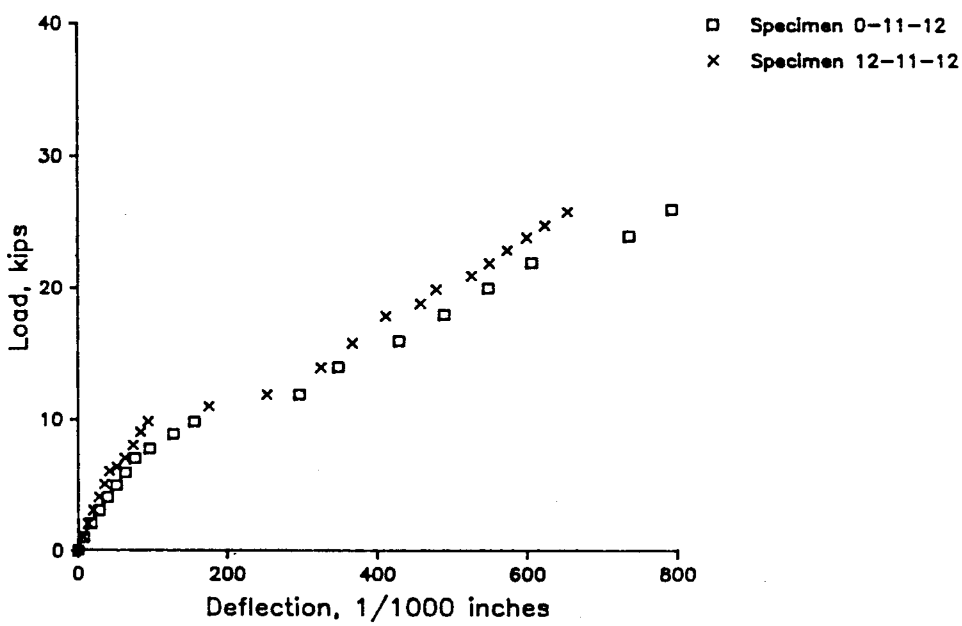


Figure 3.19 Beam End Deflection, Series X-11-12

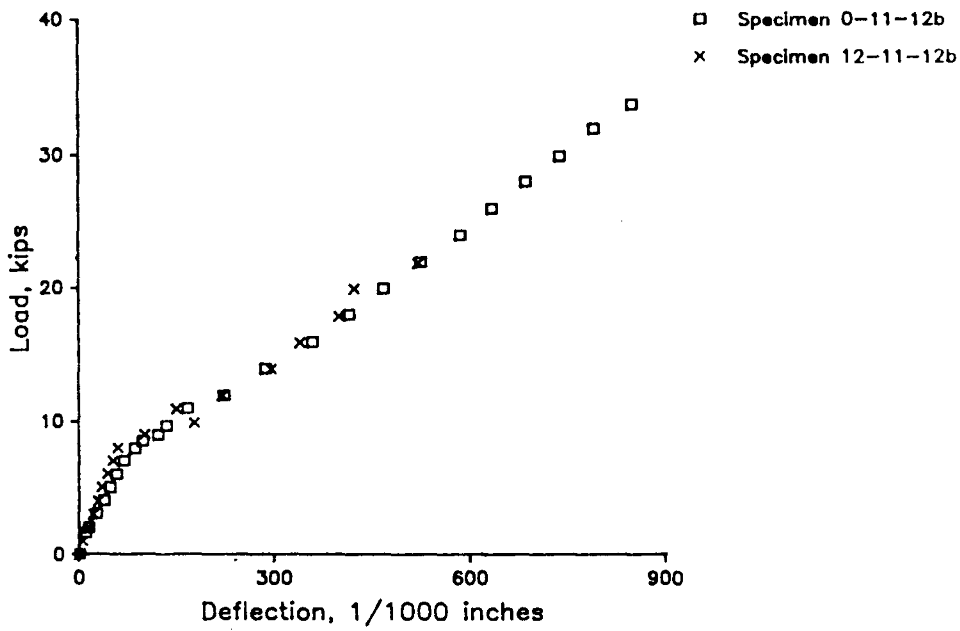


Figure 3.20 Beam End Deflection, Series X-11-12b

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

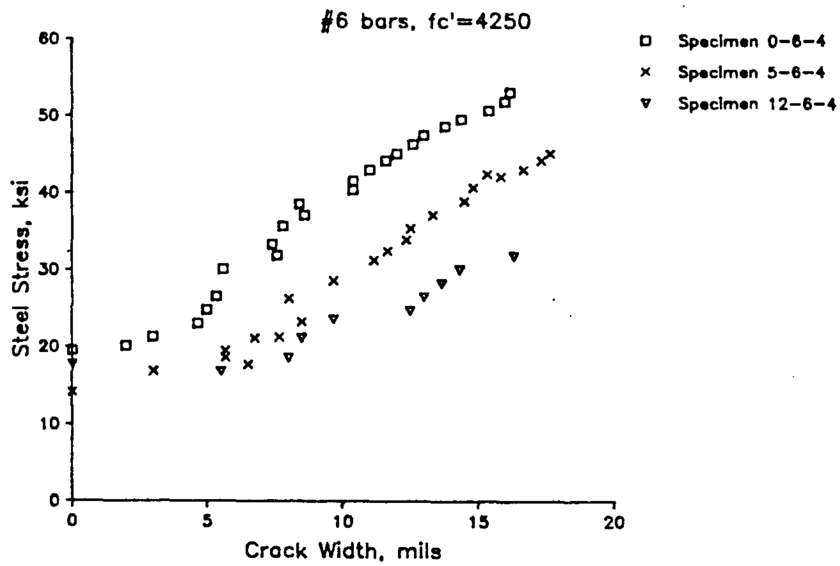


Figure 3.21 Average Crack Widths, Series X-6-4

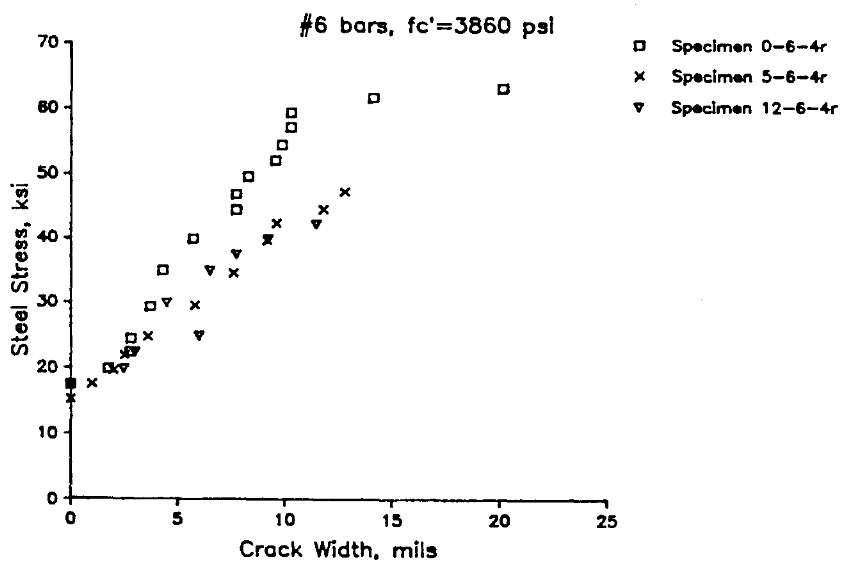


Figure 3.22 Average Crack Widths, Series X-6-4r

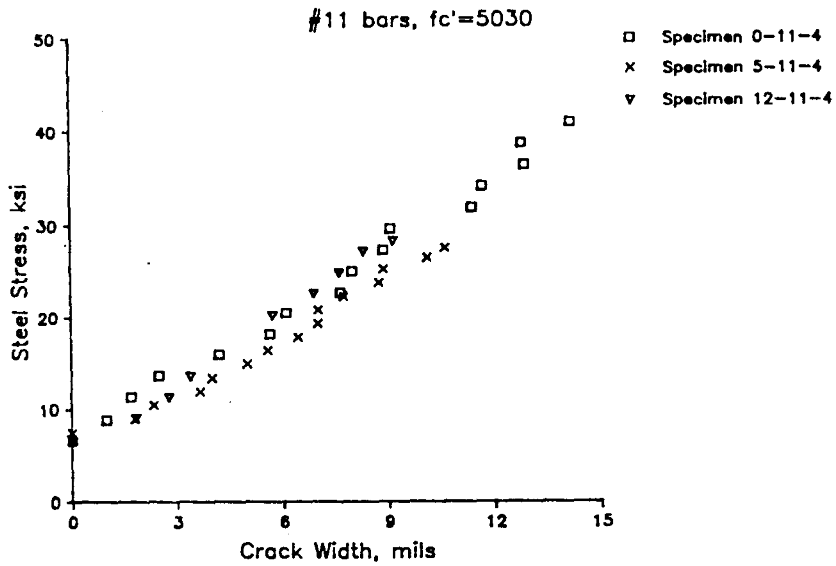


Figure 3.23 Average Crack Widths, Series X-11-4

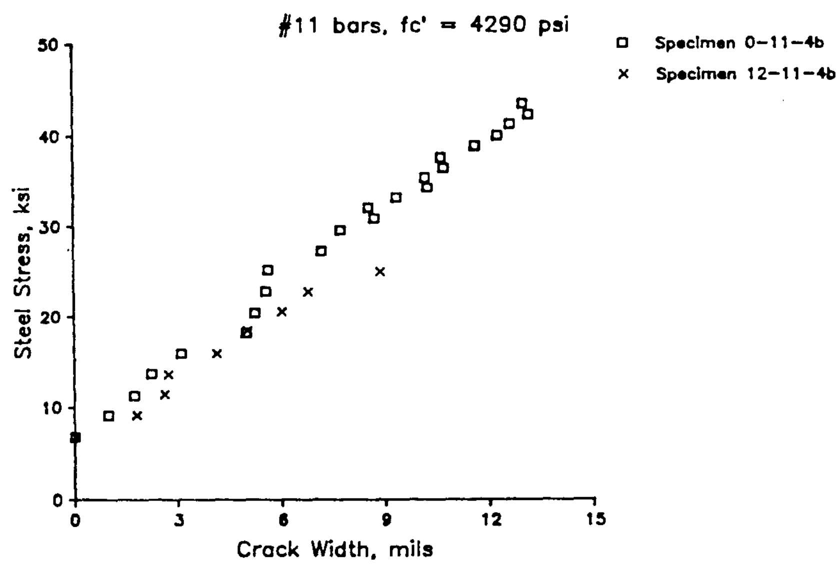


Figure 3.24 Average Crack Widths, Series X-11-4b

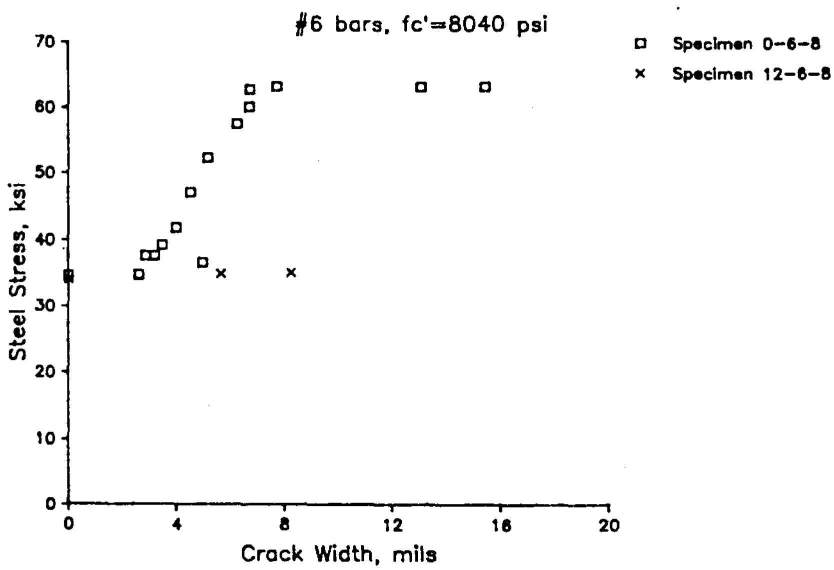


Figure 3.25 Average Crack Widths, Series X-6-8

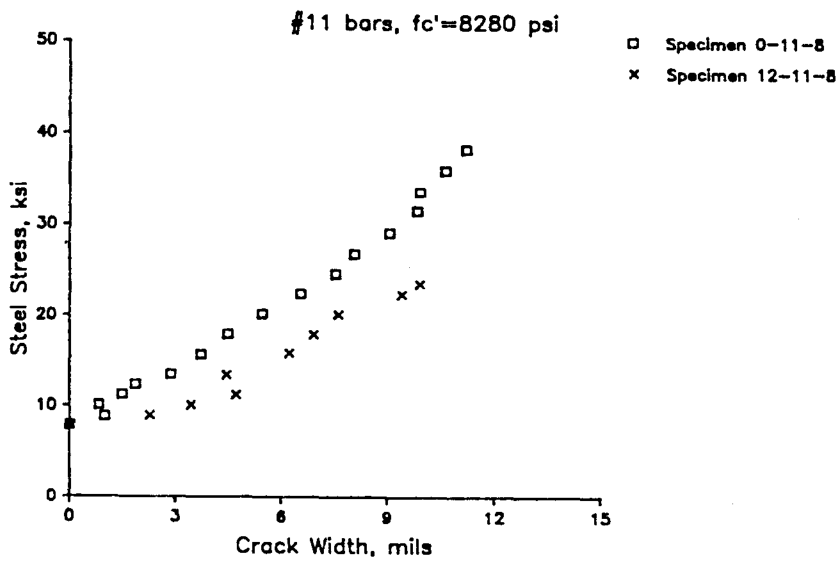


Figure 3.26 Average Crack Widths, Series X-11-8

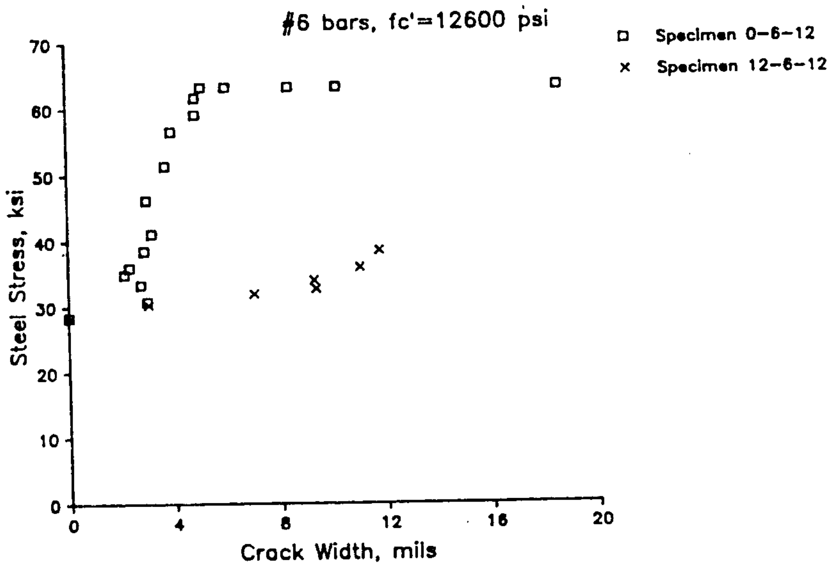


Figure 3.27 Average Crack Widths, Series X-6-12

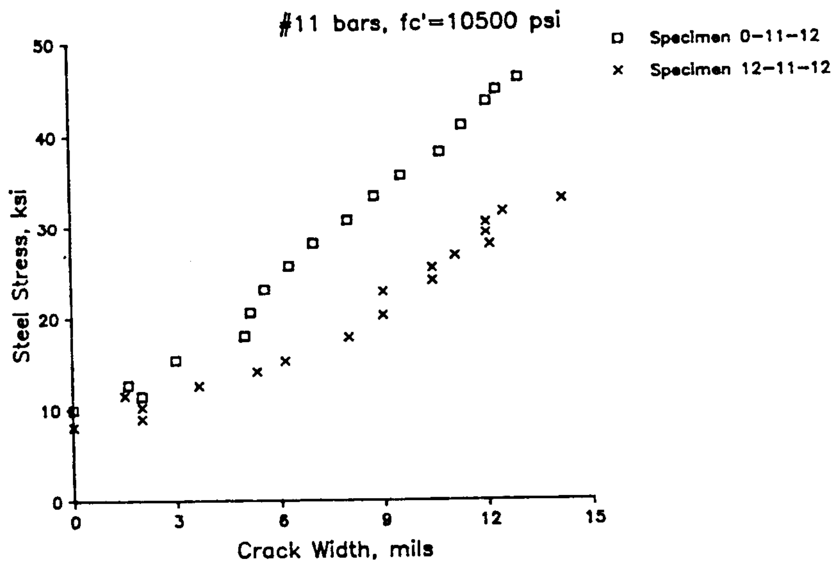


Figure 3.28 Average Crack Widths, Series X-11-12

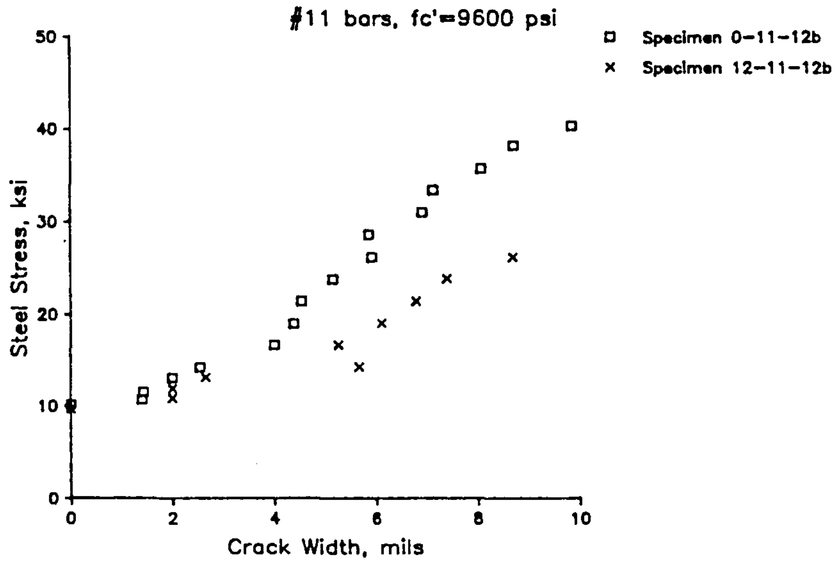


Figure 3.29 Average Crack Widths, Series X-11-12b

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

coating thickness from Table 3.4. In order to better evaluate the effect of coating thickness, the crack width ratios for each coated-bar specimen were plotted versus the average coating thickness in Fig. 3.30. A general increase in the crack width ratio is displayed as the coating thickness increases. It is difficult to draw a relationship between coating thickness and the width of cracks, since the amount of variation is so great. However, for all tests combined, the crack widths were about 50% greater for epoxy-coated bars.

3.6.2 Bar Size. It can be noted from Fig. 3.30 that the crack width ratio is generally greater for #6 bar specimens than for #11 bar specimens. The average crack width ratio for the #6 bar specimens was 2.1 while the ratio for the #11 bar specimens was only 1.4. It should be noted that the cover over the #6 bars was less than that over the #11 bars. Therefore, the influence of bar size on cracking can not be easily explained with the limited tests available.

3.7 Failure Hypothesis

The results of this program show a major difference from results of earlier studies on epoxy-coated bars. Bond strength comparisons in the NCSU study [6], showed that epoxy-coated bars developed 85% of the bond of uncoated bars. Strength comparisons of the NBS tests [5], showed that epoxy-coated bars developed 94% of the bond of uncoated bars. The results of the NBS study were influenced by the fact that most of the coated and uncoated bars yielded. The main difference between this study and the previous studies is that the bond failures in earlier tests were primarily pullout failures. All the failures in this study were caused by splitting of the cover in the splice region.

The primary reason for the reduction in bond strength appears to be the loss of adhesion between the concrete and epoxy-coated bars. As discussed in Sec. 3.2.3, there was no evidence of adhesion between the epoxy-coated bars and surrounding concrete. However, the uncoated bars showed evidence of good adhesion with the concrete. The epoxy coating breaks the bond between the steel and concrete causing most or all of the friction capacity to be lost. Friction between the concrete and steel has not been considered an important component of bond strength. The major component of bond is considered to be bearing of the deformations against the concrete. However, it was recognized by Lutz, Gergely, and Winter [12] that the

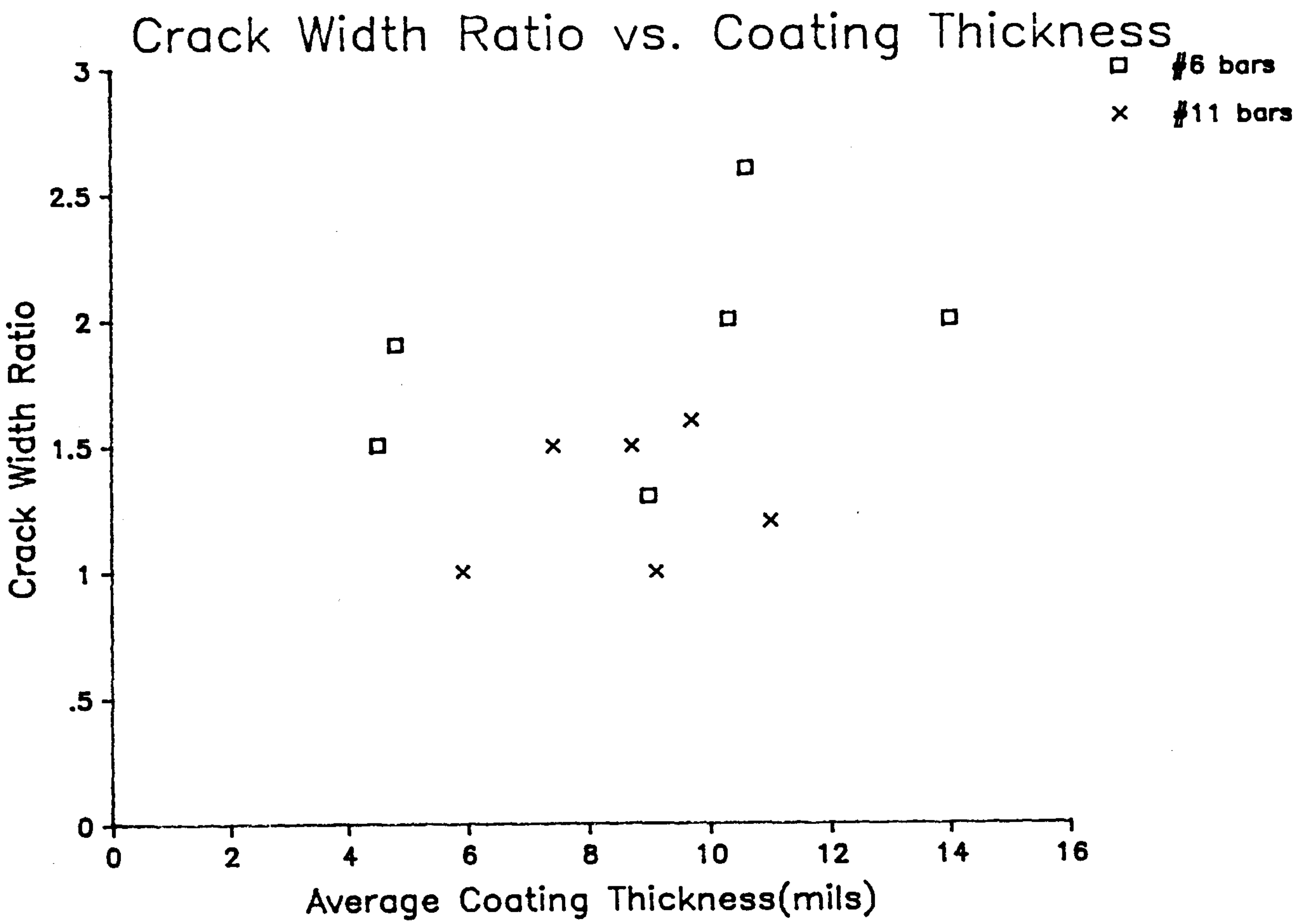

Crack Width Ratio vs. Coating Thickness
#6 bars
#11 bars
Crack Width Ratio
3
2.5
2
1.5
1
.5
0
0
2
4
6
8
10
12
14
16
Average Coating Thickness(mils)

Figure 3.30

friction between the concrete and steel at the deformations is very important in developing bond strength.

When the rib of the reinforcing steel bears against the surrounding concrete, the concrete key tends to slide up and over the face of the rib causing splitting of the concrete cover. Friction between the concrete and steel along the face of the rib acts to prevent the concrete key from sliding relative to the rib.

The force due to the friction between the steel and concrete at the rib adds vectorially to the component of bond acting perpendicular to the rib (Fig. 3.31). If the friction between the concrete and steel is lost, the only component of the bond strength is the force perpendicular to the face of the rib (Fig. 3.32).

The magnitude of the bond force is controlled by the amount of radial pressure the concrete cover can resist before splitting. This is the vertical component of the resultant bond forces in Figs. 3.31 and 3.32. The horizontal component of the resultant is the effective bond strength. If the resistance to splitting of the cover is the same for either case, then the bar with no friction will have a much smaller bond capacity than the bar which develops friction with the concrete.

In a pullout failure, the friction between the concrete and steel is much less important than in a splitting failure. A pullout failure occurs when the steel is well confined by concrete cover or transverse steel, preventing a splitting failure. In this case, the bond strength is controlled by the capacity of the concrete in direct shear. The bearing of the ribs against the concrete causes the key between ribs to shear from the surrounding concrete. Since the bar is well confined, friction between the rib and concrete is not necessary to prevent sliding of the concrete key relative to the rib.

Lutz, Gergely, and Winter predicted that bars with a larger rib face angle would be less affected by grease or other friction reducing agents than bars with a flatter rib face angle. If the face of the rib formed an angle of 90° with the axis of the bar, all of the bond strength would be produced by direct bearing of the rib against the concrete key. In this case friction between the concrete and steel would be unnecessary. However, if the bar were smooth (a rib face angle of 0°), friction between the concrete and steel would be the only component of bond. A loss of friction between the concrete and

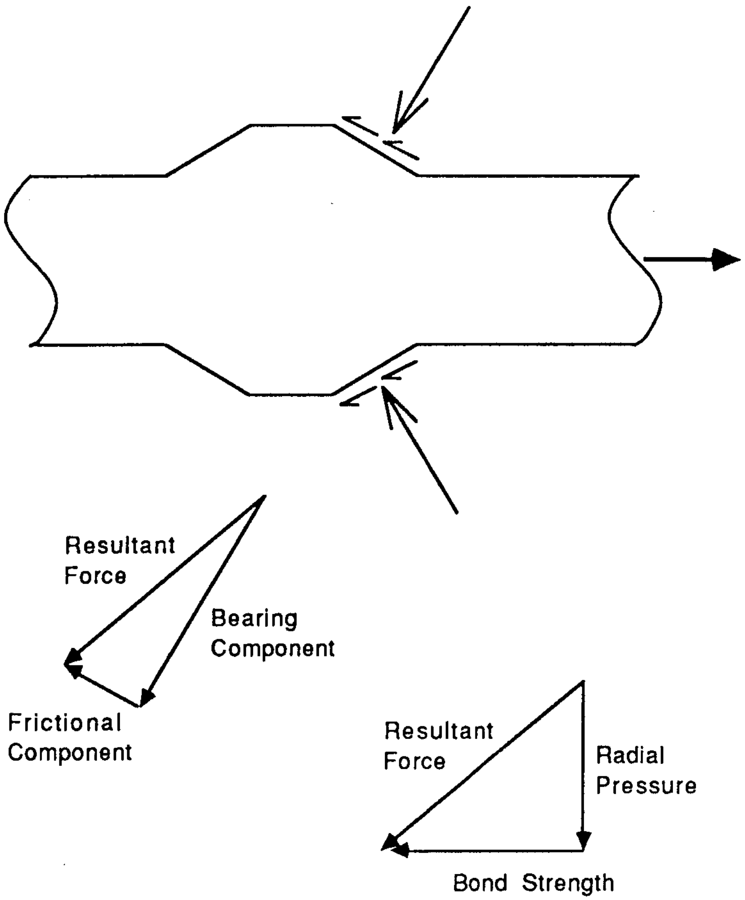


Figure 3.31 Components of Bond
With Friction

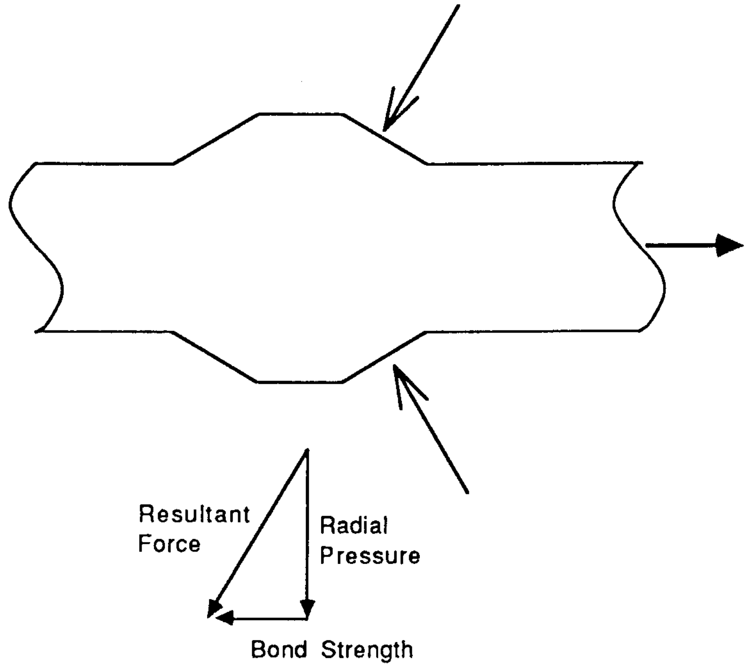


Figure 3.32 Components of Bond
Without Friction

steel would completely destroy the bond. As the rib face angle becomes larger, the component of the bearing force parallel to the face of the rib (carried by friction) decreases. Therefore the loss of friction becomes less significant.

Bars used in the current study had a rib face angle of approximately 30°. Comparison with bars produced by other manufacturers indicated that this was an average to low value. Therefore the bond strength of epoxy coated bars indicated by this study should be a conservative value. Epoxy-coated bars with a greater rib face angle should result in a higher bond strength than indicated. Lower bond strengths would be developed as the rib face angle approaches zero (plain bar).

The loss of adhesion may cause an additional reduction in bond strength by reducing the tensile capacity across the plane of splitting. Normally only concrete across the failure plane is considered to resist splitting, as shown in Fig. 3.33. However, the adhesion between uncoated bars and the surrounding concrete may cause tensile forces to develop which would increase the capacity of the cover. When the adhesion between the steel and concrete is lost due to the epoxy coating, this added splitting capacity is also lost.

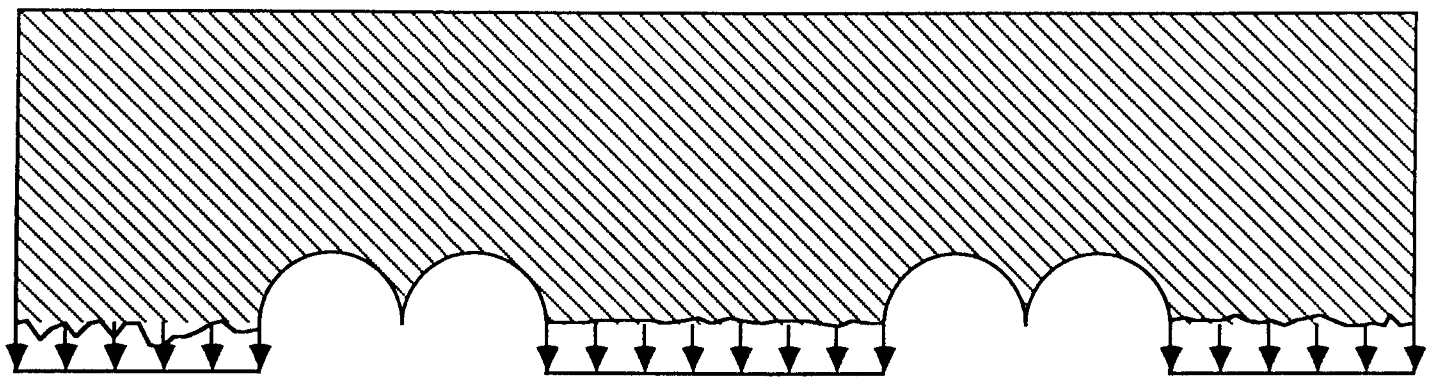


Figure 3.33 Tensile Stresses Across Splitting Plane

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

Specimen	Steel Stress (ksi)	Bond Strength (psi)	Bond Efficiency u-test/ u-theor	Bond Ratio coated/ uncoated
12-6-4	49.5	773	0.96	0.93
5-6-4	69.3	1083	1.35	1.31
0-6-4	53.1	830	1.03	1.00
12-6-4r	67.2	525	1.35	1.14
5-6-4r	71.9	561	1.57	1.33
0-6-4r	63.3	495	1.18	1.00
12-11-4	42.5	416	0.79	0.98
5-11-4	45.6	447	0.85	1.05
0-11-4	43.3	424	0.81	1.00
12-11-4b	37.4	366	0.75	0.81
0-11-4b	45.9	449	0.93	1.00
12-6-8	52.5	615	1.05	0.89
0-6-8	63.3	741	1.17	1.00
12-11-8	38.0	743	0.82	0.91
0-11-8	40.3	789	0.90	1.00
12-6-12	61.7	722	1.06	1.06
0-6-12	63.3	741	1.01	1.00
12-11-12	50.7	993	1.03	1.08
0-11-12	46.9	918	0.96	1.00
12-11-12b	41.3	808	0.88	0.96
0-11-12b	43.0	842	0.92	1.00

Table 4.1 Comparisons using adjusted bond values

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.

Two series of specimens were bottom-cast while the rest were top-cast. Because low-slump concrete was used, the results indicate virtually no difference in the bond between the top- and bottom-cast conditions. Although the test results do not define clearly the effect of casting position, applying a factor for top-cast bars as well as a factor for epoxy-coated bars seems overly conservative. If a factor of 1.5 were applied as well as the proposed 1.3 factor for top reinforcement the required development length would double. Without additional data it is unclear what effect both conditions have on the bond strength. It is likely that there is some additional effect due to top-casting, but the effect is probably not as great as for uncoated bars. Therefore, it is suggested that the combined factor for top reinforcement and epoxy-coated bars be limited to 1.7 rather than (1.5)(1.3) = 1.95 until more data becomes available.

4.4 Further Research

The current study along with previous research on epoxy-coated bars has resulted in a basic understanding of the effect of epoxy coating on bond strength. However several questions remain to be answered and further research is needed.

1. At what ratio of cover to bar diameter does a pullout failure occur and how does the bond strength of epoxy-coated bars change as the failure mode changes?

2. How effective is epoxy-coated transverse reinforcement in providing confinement to both epoxy-coated and uncoated longitudinal reinforcement?

3. What is the effect of top-casting (with high-slump concrete and a large depth of concrete below the bars) on the reduction in bond strength due to epoxy coating?

4. What is the effect of epoxy coating on bars with a large rib face angle and different deformation patterns?

5. What is the effect of epoxy coating on various sizes of bars?

Future research should be concentrated on tests resulting in a splitting failure. The current study shows that the reduction in bond is much more significant than for a pullout failure. Splitting failure tests will result in more meaningful information on the bond strength of epoxy-coated bars. Beam 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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