ACI MATERIALS JOURNAL TECHNICAL PAPER

Title no. 86-M17

A Research Report Sponsored by the
Reinforced Concrete Research Council

Bond Strength of Epoxy-Coated Reinforcing Bars


by R. A. Treece and J. O. Jirsa

Epoxy-coated bars are used in nearly all types of structures where corrosion may cause deterioration. Bridge decks and parking garages are especially susceptible to salt-induced damage. For satisfactory structural performance, bond between concrete and steel is essential, and the effect of coatings that might decrease bond should be considered in designs.

In this study, 21 beams with lap splices in a constant moment region were tested in nine groups and the bond strength of epoxy-coated bars was compared to that of uncoated bars. Variables were bar size, concrete strength, casting position, and coating thickness. In each test group, the only variable was the coating thickness.

Keywords: anchorage (structural); **bond (concrete to reinforcement); coatings;** deformed reinforcement; **epoxy resins;** high-strength concretes; lap connections; **reinforcing steels;** splicing.

Epoxy-coated bars are used to provide protection against corrosion, which leads to premature deterioration of concrete structures. A primary use is in bridge decks where corrosion due to deicing salts may occur. However, they are used in nearly all types of structures. Parking garages are especially susceptible to salt-induced damage because it is difficult to provide adequate drainage for the floors. Elements of a structure adjacent to traveled roadways are exposed to salt-laden spray from trucks. In coastal regions, all elements of a bridge exposed to seawater or sea spray may be built with epoxy-coated bars. Other applications include sewage treatment plants, water-chilling stations, and chemical plants.

Due to the importance of development and splices of reinforcement in analysis and design of reinforced concrete structures, bond between concrete and steel is essential. Coatings that might decrease bond should not be applied to reinforcing bars. ACI 318-86, Section 7.4.1[1] states that bars should be free of nonmetallic coatings, mud, or oil that may decrease the bond capacity. Epoxy coatings, however, have been used for over 10 years and are permitted by Section 3.5.3.7. The Commentary to Section 3.5.3.7. cautions the designer about the bond performance of epoxy-coated bars, especially "in conditions where they are subjected to cyclic loads or minimum development lengths or anchorages."

PREVIOUS RESEARCH

A series of 28 #6 bars were embedded in large concrete prisms and subjected to concentric pullout tests by Mathey and Clifton.[2] Twenty-three bars had varying coating thicknesses, and different methods of coating application were used. Five bars were uncoated. In most tests, coating thicknesses ranged from 1 to 11 mil (1 mil = 0.025 mm) but two bars had a coating thickness of 25 mil. Large concrete prisms provided adequate confinement to prevent splitting failures. However, the concrete prism was in compression at the loading surface and does not represent the actual condition where the concrete is in tension.

Based on a comparison of critical bond strengths, it was concluded that bars with a coating thickness from 1 to 11 mil developed acceptable bond strengths. Mathey and Clifton stated that, "The average value of applied load corresponding to the critical bond strength in the 19 pullout specimens with bars having epoxy coatings 1 to 11 mil thick was 6 percent less than for pullout specimens containing the uncoated bars." The critical bond strength in their studies refers to the lesser of the bond stresses corresponding to a loaded-end slip of 0.01 in. (0.25 mm) or to a free-end slip of 0.002 in. (0.05 mm) and does not represent the ultimate bond strength of the bar. Two different bar deformation patterns (not necessarily from the same heat of steel) were used. Comparisons of critical bond strengths were made between two groups of randomly selected bars. In

ACI Materials Journal, V. 86, No. 2, March-April 1989.
Received May 16, 1988, and reviewed under Institute publication policies.

Pertinent discussion will be published in the January-February 1990 *ACI Materials Journal* if received by Oct. 1, 1989.

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addition, all of the uncoated bars as well as the coated bars with 1 to 11 mil coating thicknesses yielded in the tests. Bond failures occurred in only two epoxy-coated bars: those with a coating thickness of 25 mil. It was recommended that bars with an epoxy coating thickness greater than 10 mil not be used.

Johnston and Zia[3] reported tests of slab specimens that were used to compare strength, crack width, and crack spacing. Beam-end specimens with coated and uncoated #6 and #11 bars were tested under both static and fatigue loadings. The slab specimens showed little difference in crack width and spacing, deflections, or ultimate strengths between coated and uncoated bars. The epoxy-coated bar specimens failed at approximately 4 percent lower loads than those with uncoated bars. However, most of the slabs failed in flexure rather than in bond.

The beam-end specimens were flexural-type specimens in which load was applied to the reinforcing bar. Splitting occurred along the reinforcing bars but the primary modes of failure were either pullout or yielding of the reinforcing steel. Some tests were terminated after yielding but before a pullout failure occurred. Based only on tests that ended in a pullout failure, the uncoated bars developed 17 percent more bond strength than the epoxy-coated bars. This corresponds to the epoxy-coated bars developing about 85 percent of the bond of uncoated bars. Results of the fatigue tests showed similar results. To account for the reduction in bond strength due to epoxy coating, it was recommended that the development length be increased by 15 percent when using epoxy-coated reinforcing bars.

EXPERIMENTAL PROGRAM

In this study, 21 beams were tested and the bond strength of epoxy-coated bars was compared to that of uncoated bars. Variables were bar size, concrete strength, casting position, and coating thickness. In each of nine series, a different combination of variables was examined, but the only variable within a series was the coating thickness.

Each series included a specimen with uncoated bars and a specimen with bars having a 12-mil coating. In some series, a third specimen with bars having a coating thickness of 5 mil was cast. The minimum and maximum (5 and 12 mil) coating thicknesses are specified by ASTM A 775-84.[4] Specimens were cast with either #6 or #11 bars. Three nominal concrete strengths, 4, 8, and 12 ksi (28, 55, and 83 MPa), were used. Seventeen specimens were cast with bars in the top position [12 in. (300 mm) of concrete below bars] and four specimens were bottom cast. Test parameters for each specimen are shown in Table 1.

TEST SPECIMENS

Test specimens were beams with three bars in tension, all spliced at the center. The splice lengths were established so that the bars would fail in bond before

Table 1 — Details of test specimens

Specimen notation*	ℓ_s, in.	d_b, in.	c_b, in.	f'_c, ksi	Coating thickness, mil: Average	Coating thickness, mil: Standard deviation	Measured maximum strength, ksi
12-6-4	12	0.75	2	4.25	10.6	2.0	33.0
5-6-4	12	0.75	2	4.25	4.8	2.1	46.2
0-6-4	12	0.75	2	4.25	0		53.1
12-6-4r†	24	0.75	7/8	3.86	9.0	2.1	44.8
5-6-4r	24	0.75	3/4	3.86	4.5	1.4	47.9
0-6-4r	24	0.75	1	3.86	0		63.3
12-11-4	36	1.41	2	5.03	9.1	2.8	28.3
5-11-4	36	1.41	2	5.03	5.9	1.9	30.4
0-11-4	36	1.41	2	5.03	0		43.3
12-11-4b‡	36	1.41	2	4.29	11.0	3.9	24.9
0-11-4b	36	1.41	2	4.29	0		45.9
12-6-8	16	0.75	3/4	8.04	14.0	3.3	35.0
0-6-8	16	0.75	7/8	8.04	0		63.3
12-11-8	18	1.41	2 1/4	8.28	7.4	2.4	25.3
0-11-8	18	1.41	2 1/8	8.28	0		40.3
12-6-12	16	0.75	5/8	12.60	10.3	3.3	41.1
0-6-12	16	0.75	3/4	12.60	0		63.3
12-11-12	18	1.41	2	10.51	9.7	2.5	33.8
0-11-12	18	1.41	2	10.51	0		46.9
12-11-12b	18	1.41	2	9.60	8.7	2.6	27.5
0-11-12b	18	1.41	2	9.60	0		43.0

*First number is nominal coating thickness, second is bar size, and third is nominal f'_c in ksi.
†Repeat test.
‡Bottom cast; all others top cast.
Note: 1 in. = 2.54 cm; 1 ksi = 6.9 MPa; 1 mil = 0.025 mm.

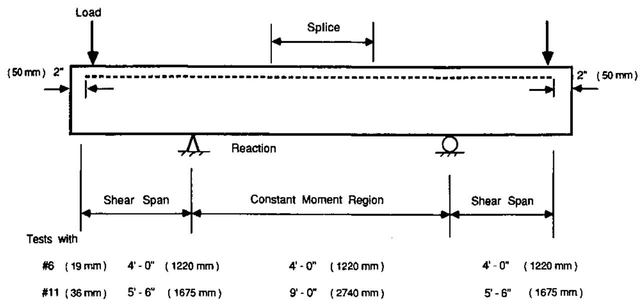

Fig. 1—Test setup and beam dimensions

reaching yield, based on an empirical equation developed by Orangun, Jirsa, and Breen.[5] The specimens were tested in negative bending with a constant moment region in the middle of the specimen (Fig. 1). With the tensile surface on top, marking and measuring cracks was easier.

The specimens were originally designed with 2 in. (50 mm) of cover on the side and top faces. The clear spacing between splices was 4 in. (100 mm). The top cover on the specimens with #6 bars was later changed to ¾ in. (19 mm) to allow a longer splice length without developing yield in the bars. No transverse reinforcement was provided in the splice region so that splitting rather than a pullout would govern failure. Specimen dimensions are shown in Fig. 1 and 2.

All bars of the same size were from the same heat and had a diamond deformation pattern. The thickness of the epoxy coating was measured and the average coating thickness (and standard deviation) for the coated bars in each specimen is shown in Table 1. Measured thicknesses varied significantly from the average values, as indicated in Fig. 3, which is typical of the measurements of epoxy coating.

Construction and test procedure

All beams in a series were cast from the same batch of concrete. The concrete was placed in two lifts and compacted with mechanical vibrators.

Load was applied to the specimens with two 60 kip (270 kN) rams at each end. Load increments of about 1 kip (4.5 kN) were applied until the beam was cracked over the constant moment region. Subsequently, load was applied at increments of about two kips. At each load stage, cracks were marked and crack widths were measured. Deflections were read at the load point and at the center of the beam.

TEST RESULTS

General behavior

In specimens with #11 bars, 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 typical of splices where the top and side cover are equal.[5] In specimens with #6 bars, 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

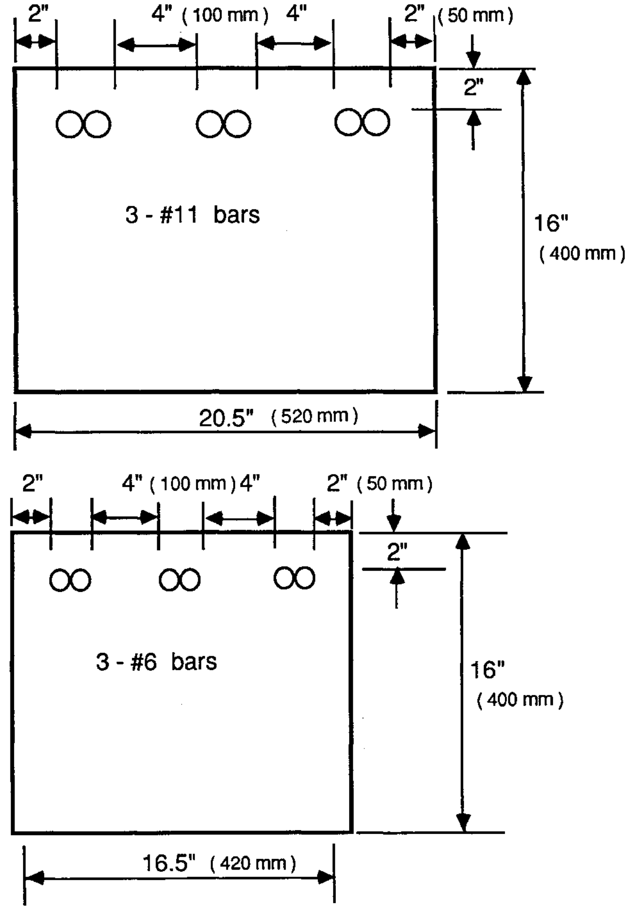

Fig. 2—Beam cross sections

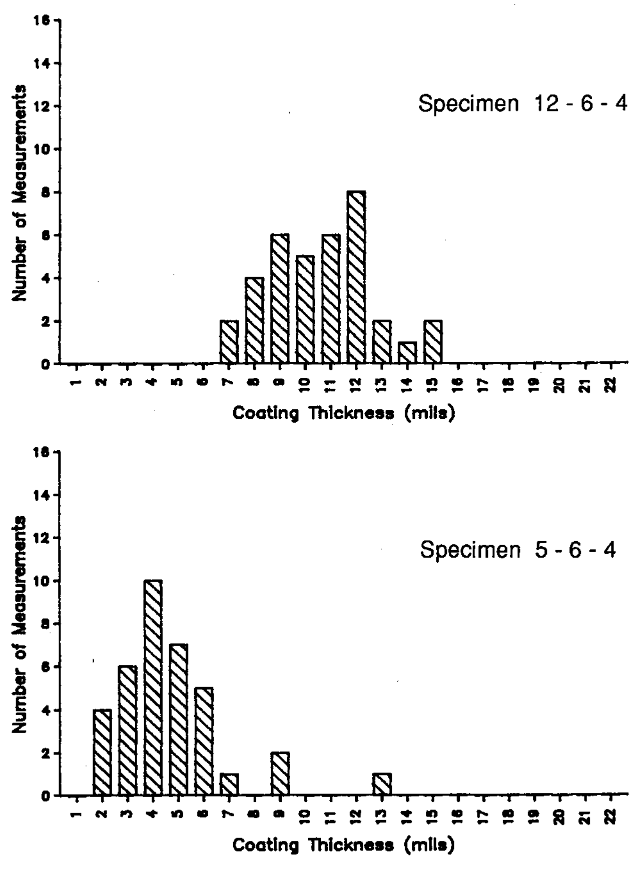

Fig. 3—Distribution of coating thickness measurements

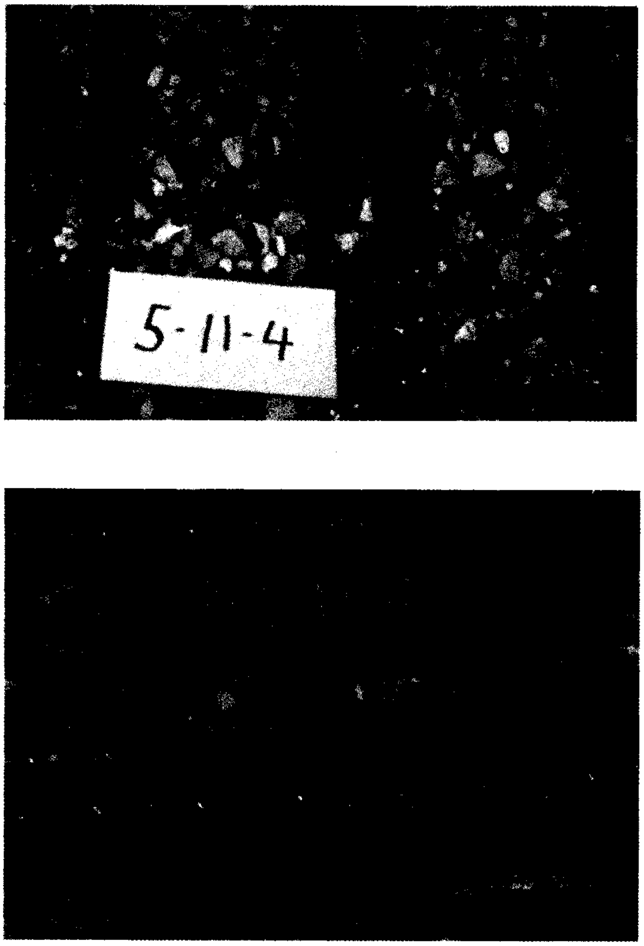

Fig. 4—Appearance of concrete cover and coated bars after test

V-notch failure typical of splices where the side cover and spacing between splices are much greater than the top cover.[5] Longitudinal cracking in specimens with coated bars was followed by a splitting failure with little increase in the load. Longitudinal cracking in the specimens with uncoated bars was followed by a significant increase in the load before failure occurred.

After the test was completed, the top cover over the splice was removed to study 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 (Fig. 4). 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 (Fig. 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. 5). The concrete cover in contact with the bars was dull and rough. Pieces of mill scale were removed from the bars and were still in contact with the concrete. There was crushing of the concrete due to bearing against the bar lugs.

Bond strength

In each test, the mode of failure was a splitting failure at the splice region. Therefore, the bond strength u 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. From equilibrium $A_s f_s = u\pi\, d_b \ell_s$ and solving for u gives $u = f_s d_b / 4\ell_s$.

The steel stress developed in each specimen (Table 1) was determined by analyzing the section based on cracked, elastic behavior, ignoring the tensile stresses in the concrete below the neutral axis. Compressive stresses in the concrete were assumed to vary linearly with distance from the neutral axis.

In three specimens, 0-6-4r, 0-6-8, and 0-6-12, the uncoated bars yielded before a splitting failure was reached. However, this did not affect significantly the bond strength calculated because splitting failure occurred shortly after the bars yielded. The bar stress was taken as the measured yield strength, 63.3 ksi (436 MPa).

The equation developed by Orangun, Jirsa, and Breen[5] was used to determine theoretical bond strength. The equation includes the cover c, concrete strength f'_c, bar diameter d_b, splice length ℓ_s, and c, the lesser of the clear cover or half clear spacing between bars. For bars with no transverse reinforcement providing confinement:

$$u/\sqrt{f'_c} = [1.2 + 3(c/d_b) + 50(d_b/\ell_s)] \qquad \text{Eq. (1)}$$

The measured bond strength for each specimen was divided by its theoretical bond strength to obtain a bond efficiency. The bond strength using ACI 318 was also computed and compared with measured values. In computing u_{ACI} values, ACI 318 factors for top bar effect (1.4) and for class of splice (factor for Class C is 1.7) were not considered. The top bar factor (1.3) suggested for use with Eq. (1) also was omitted. Splices and development length are given by the same equation [Eq. (1)] in Reference 5. 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 to obtain a bond ratio. The bond strengths, bond efficiencies, and bond ratios for each specimen are shown in Table 2. The specimens in which the bars yielded are denoted with a *Y* next to the bond efficiency. The bond ratios show a significant reduction in bond due to the epoxy coating. The average ratio of coated to uncoated strength is 0.67 with a standard deviation of 0.09. It can also be seen that the equation from Reference 5 is considerably more accurate than the ACI 318 approach for the bond strength of uncoated bars. The ratio u_{test}/u_{ACI} is 1.23 and u_{test}/u_{theor} is 0.99. If the Class C splice factor of 1.7 is used in computing bond efficiency, the value of u_{test}/u_{ACI} is 2.15 (standard deviation of 0.40) for the uncoated bars.

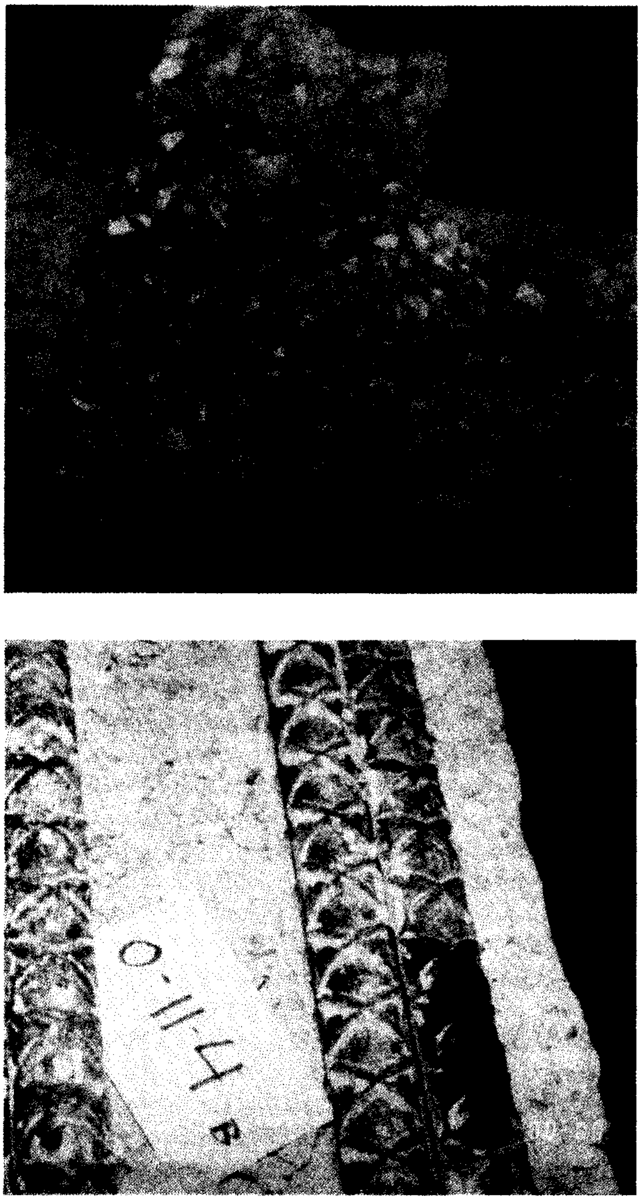

Fig. 5—Appearance of concrete cover and uncoated bars after test

Table 2 — Comparison of results

Specimen	Measured bond strength, psi u_{test}	Computed bond strength, psi u_{ACI}*	Computed bond strength, psi u_{theor}	Bond efficiency u_{test}/u_{ACI}*	Bond efficiency u_{test}/u_{theor}	Bond ratio coated/uncoated
12-6-4	520	690*	800	0.83	0.64	0.62
5-6-4	720	690*	800	1.15	0.90	0.87
0-6-4	830	690*	800	1.33	1.03	1.00
12-6-4r	350	660*	390	0.56	0.90	0.71
5-6-4r	370	660*	360	0.59	1.05	0.76
0-6-4r	500 Y	660*	420	0.80	1.18	1.00
12-11-4	280	400	530	0.70	0.53	0.65
5-11-4	300	400	530	0.75	0.57	0.70
0-11-4	420	400	530	1.05	0.81	1.00
12-11-4b	240	370	490	0.65	0.50	0.54
0-11-4b	450	370	490	1.22	0.93	1.00
12-6-8	410	960*	590	0.66	0.70	0.55
0-6-8	740 Y	960*	630	1.18	1.17	1.00
12-11-8	500	520	900	0.96	0.55	0.61
0-11-8	790	520	880	1.52	0.90	1.00
12-6-12	480	1200*	680	0.77	0.71	0.65
0-6-12	740 Y	1200*	740	1.18	1.01	1.00
12-11-12	660	580	960	1.14	0.69	0.72
0-11-12	920	580	960	1.58	0.96	1.00
12-11-12b	540	560	920	0.99	0.59	0.64
0-11-12b	840	560	920	1.52	0.92	1.00
Average of all coated bars:				0.81	0.69	0.67
S.D.:				0.19	0.17	0.09
Average of all uncoated bars:				1.23	0.99	
S.D.:				0.25	0.12	

Y — Bar yielded.
*Upper limit on bond stress is 625 psi.
Note: 1000 psi = 6.9 MPa.

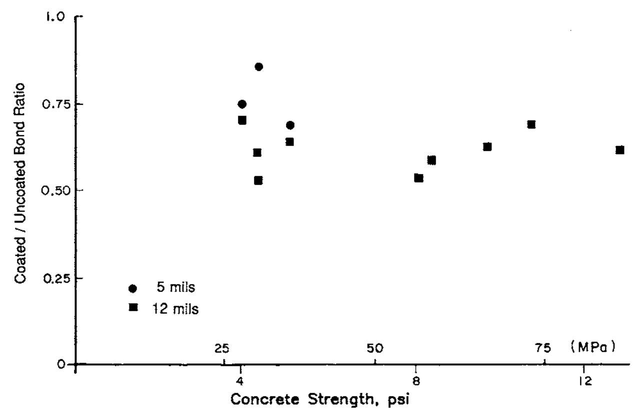

Fig. 6—Bond ratio versus concrete strength for coated bars

Fig. 6 shows the bond ratio as a function of concrete strength. The bond ratio for the bars with nominal coating thickness of 12 mil is between 0.54 and 0.71 for all concrete strengths. It appears that the reduction in bond due to the epoxy coating does not vary with the concrete strength.

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 low-slump [less than 4 in. (100 mm)] concrete was used. The quality of bond in top-cast series was not significantly less than the bond in corresponding bottom-cast series, and the bond strength of the uncoated bars was not affected by the casting position.

Recommendations made by Jirsa and Breen[6] on the effect of casting position on bond showed that for low-slump concrete the ACI 318 casting position factor is very conservative. The casting position factor from Reference 6 for the low-slump concrete used in this program is only 1.06.

Fig. 6 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 suggest that the bond reduction is less for a small coating thickness. The actual coating thicknesses, however, varied significantly from the nominal values of 5 and 12 mil, as can be seen by the distribution of coating thicknesses in Fig. 3. The bond ratio for the coated bar specimens is plotted against the average coating thickness of the bars in Fig. 7. To show the variation in the coating thickness, one standard deviation above and below the mean is also plotted.

The two specimens with the smallest coating thicknesses had higher bond ratios than the other coated bar

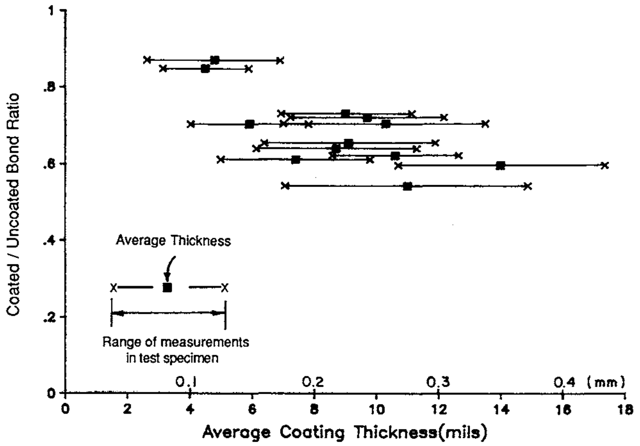

Fig. 7—Bond ratio versus coating thickness

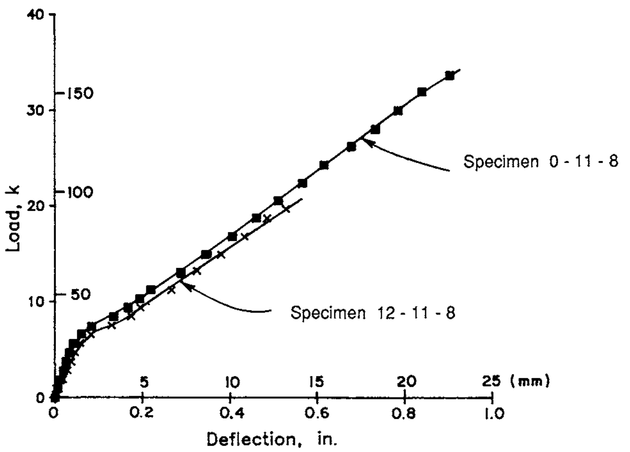

Fig. 8—Load versus beam end deflection, Series X-11-8

specimens. If these two specimens with an average coating thickness of 4.5 and 4.8 mil were excluded, virtually no variation in the bond reduction with coating thickness could be detected. Note that ASTM requires a minimum average coating thickness of 5 mil and a maximum average coating thickness of 12 mil. The specimen with the average coating thickness of 14 mil follows the general trend of the specimens with coating thicknesses between the limits of 5 and 12 mil.

In summary, there was virtually no variation in bond strength except between coated and uncoated bars. The bond ratio for epoxy-coated bars with average coating thicknesses above 5 mil was 0.67 with a standard deviation of 0.09. The reduction in bond was consistent for the range of variables considered in this study.

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. A typical load-deflection curve is shown in Fig. 8. Little difference in stiffness was noted between specimens with uncoated and coated bars.

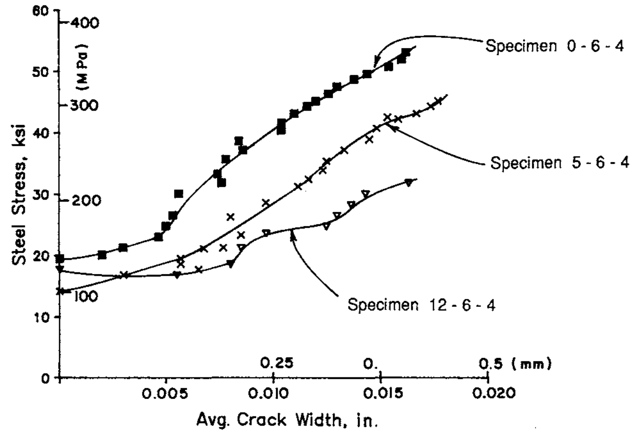

Fig. 9—Average crack widths in constant moment region outside of splice, Series X-6-4

Crack width and spacing

The cracks outside the splice length in the constant moment region represent most accurately the effect of epoxy coating on the spacing and width of cracks. The crack widths outside the splice length were averaged and plotted versus steel stress for one series of specimens (Fig. 9). In general, the specimens with epoxy-coated bars exhibited wider average cracks than the uncoated bar specimens. Specimens with epoxy-coated bars had fewer cracks (wider spacing), but the width of the cracks was greater than in uncoated bar specimens. The loss of adhesion due to coating resulted in a longer length of bar required to transmit stresses from the bar to the concrete to produce a flexural crack.

Details of the tests and results are given in Reference 7.

FAILURE HYPOTHESIS

The test results show a major difference from results of earlier studies on epoxy-coated bars. Bond strength comparison in Reference 3 showed that epoxy-coated bars developed 85 percent of the bond of uncoated bars. Strength comparisons in Reference 2 showed that epoxy-coated bars developed 94 percent of the bond of uncoated bars, but most of the coated and uncoated bars yielded. The main difference between this and previous studies is that 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 and surrounding concrete. The epoxy coating destroyed adhesion between the steel and concrete, causing most or all of the friction capacity to be lost. In contrast, the uncoated bars showed evidence of good adhesion with the concrete. Friction between the concrete and steel generally 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[8] that the friction between the concrete and steel at the deformations is important in developing the bond strength.

When the rib of the bar bears against the surrounding concrete, the concrete key tends to slide up 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. 10). 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.

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 Fig. 10. The horizontal component of the resultant is the effective bond strength. If the capacity of the cover is the same for either case, the bar with no friction will have a much smaller bond capacity than the bar that develops friction between the concrete and the bar lug.

In a pullout failure, friction between the concrete and steel should be 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 should be controlled primarily by the capacity of the concrete in direct shear. 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[8] 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 deg 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, for a plain bar (rib face angle of 0 deg), friction caused by adhesion between the concrete and steel would be the only component of bond. Loss of adhesion between the concrete and 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. Additional work is needed to clarify the importance of adhesion and rib face angle on bond.

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. 11. However, the adhesion between uncoated bars and the surrounding concrete may cause tensile forces to develop that would increase the capacity of the cover. When the adhesion between the steel

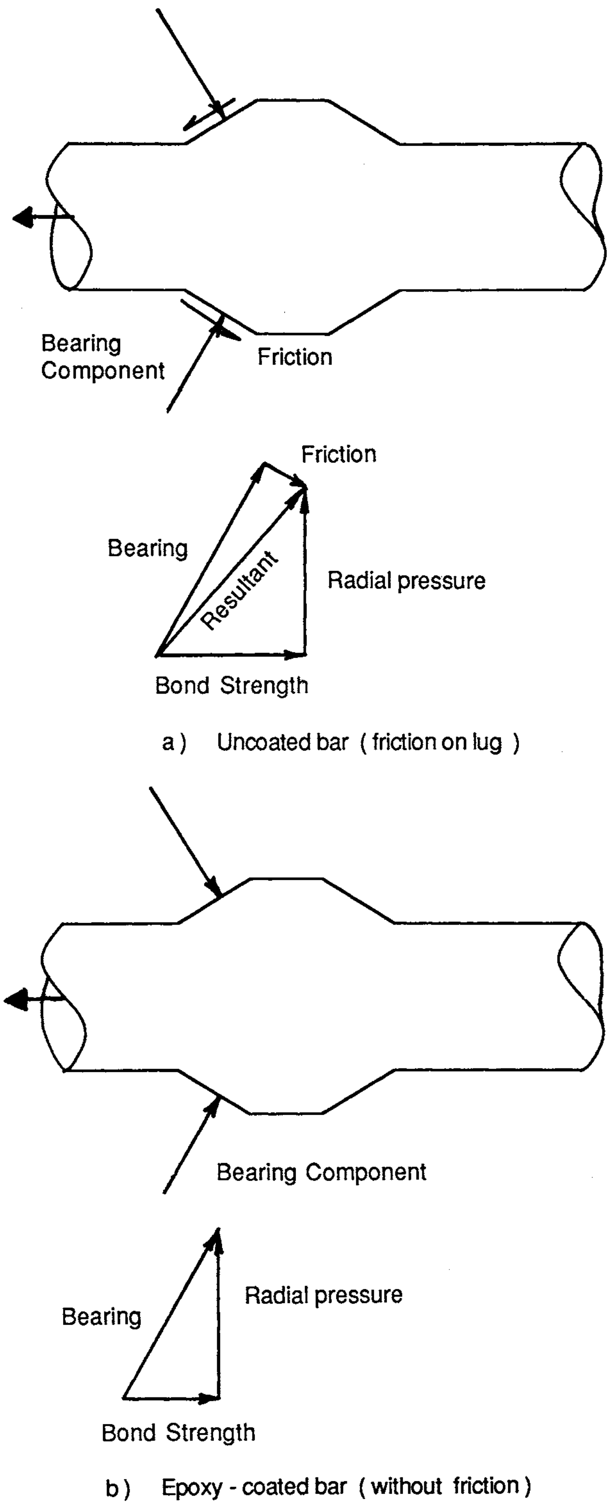

Fig. 10—Bond strength components

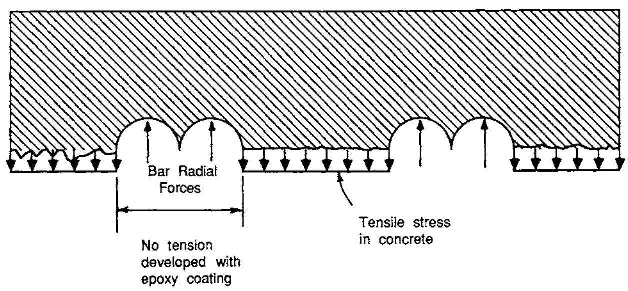

Fig. 11—Tensile stresses across splitting plane

and concrete is lost due to the epoxy coating, this added splitting capacity is also lost.

DESIGN RECOMMENDATIONS

Tests indicate that the development or splice length must be increased when using epoxy-coated reinforcing bars. The amount of increase is dependent on the type of bond failure that will occur. All the tests in the current study resulted in a splitting failure with a reduction of about 35 percent in bond strength for coated bars. Previous studies on epoxy-coated bars showed that the reduction in bond (6 to 15 percent) is much less for a pullout failure.

For coated bars to develop the same capacity as uncoated bars, the development length should be increased by the reciprocal of the bond ratio. A 15 percent increase in the development length for epoxy-coated bars was recommended in Reference 3. This value is considered appropriate for bars with large cover or wide spacing where splitting is unlikely. Based on an average measured bond ratio of 0.67 for the tests reported herein, the development length should be increased by a factor of 1.5 for epoxy-coated bars with small cover or close spacing where splitting is likely.

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

One area that needs to be studied in much greater detail is the influence of transverse reinforcement on the bond strength of epoxy-coated bars. In a splice or development length well confined by transverse reinforcement, a splitting failure can be prevented and the effect of the epoxy coating should 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 probably is less than that provided by uncoated transverse steel.

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. The amount of the reduction was dependent on the mode of failure: pullout or splitting.
2. If a splitting failure occurred, the bond strength of epoxy-coated bars was approximately 65 percent of the bond strength of uncoated bars. If a pullout failure occurred, the bond strength was approximately 85 percent of that for uncoated bars.
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 mil and less than about 14 mil.
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 coatings.

ACKNOWLEDGMENTS

The research study was sponsored by the Concrete Reinforcing Steel Institute and the Reinforced Concrete Research Council under Project 50. The advice and comments of Task Group Chairman John F. McDermott and members David P. Gustafson, Roger H. Wildt, Henry G. Russell, Robert J. Smith, and Paul Zia were sincerely appreciated. Mr. B. K. Thornley of Florida Steel Corporation was most helpful in providing the reinforcing bars used in the study. The careful attention of Mr. V. A. Walther, Jr., of CRSI, to details associated with the support of the project helped make the project proceed rapidly.

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