Bond Characteristics of High-Strength Steel Reinforcement

by Raafat El-Hacha, Hossam El-Agrousy, and Sami H. Rizkalla

This paper summarizes an investigation undertaken to study the bond characteristics of high-strength steel reinforcement bars commercially known as microcomposite, multistructural, formable steel (MMFX). The objective of the investigation is to examine the applicability of the ACI 318-02 equation and a current proposed equation by Zuo and Darwin on bond behavior of steel reinforcement to the concrete member. The experimental program included two phases. The first phase of the experimental program consisted of testing four beam-end specimens reinforced with MMFX steel bars, whereas the second phase included testing eight beam-splice specimens reinforced with MMFX steel bars. The selected four factors considered in this study were bar size, level of confinement, bonded length, and bar cast position. The bond behavior of the MMFX steel bars was found to be similar to that of conventional Grade 420 MPa (60 ksi) steel up to the proportional limit of 550 MPa (80 ksi). The bond strength of the MMFX significantly changes as the tensile stresses developed in the bar exceed the proportional limit. The test results indicated that both the ACI 318-02 equation and the current proposed equation by Zuo and Darwin on bond are adequate and resulted in conservative prediction at low stress levels up to 550 MPa (80 ksi). At high stress levels, however, the prediction using both equations is unconservative due to the nonlinear behavior of the MMFX stress-strain relationship. Based on the limited number of specimens considered in this study, modification to both the ACI 318-02 equation and the Zuo and Darwin equation is proposed to predict the bond forces beyond the proportional limit for MMFX steel bars.

Keywords: bond; confinement; flexure; splice; steel; strength.

INTRODUCTION

Bond behavior of reinforcing steel bars to concrete is one of the most important mechanisms that should be properly designed to ensure satisfactory performance of reinforced concrete structures. The bond strength and mode of bond failure are affected by many factors. The most important factors are thickness of the clear concrete covers (bottom and/or side), clear spacing between bars, nominal bar diameter, embedment or development and splice length, amount of transverse steel reinforcement, and concrete compressive strength. The individual contributions of these factors are difficult to separate or quantify. Another factor that influences the bond strength of bars is the depth of fresh concrete below the bar during casting.

In general, any increase in confinement of the bar by the surrounding concrete, and/or by transverse reinforcement increases the bond strength and minimizes splitting. Confinement by the concrete is dependent on the clear concrete covers (bottom and/or side) and the bar spacing. Increasing the development/splice length of a reinforcing bar increases its bond strength. The bond strength, for a given length, mobilized by both concrete and transverse reinforcement, increases as the bar diameter increases. Bond strength of bars confined by transverse reinforcement increases with the increase in the relative rib area. Top-cast bars have lower bond strength than bottom-cast bars. Also, bond strength increases with increasing concrete compressive strength for bars not confined by transverse reinforcement approximately with the 1/4 power of the compressive strength ($f_{c}^{1/4}$). The additional bond strength, provided by transverse reinforcement, increases approximately with the 3/4 power of the compressive strength ($f_{c}^{3/4}$). Note that the $f_{c}^{1/4}$ has been shown to provide a better representation of the effect of concrete strength on bond than $f_{c}^{1/2}$. This point is recognized by ACI Committee 408 and within ACI 318, which sets an upper limit on the value of $f_{c}^{1/2}$ for use in design. An increase in the aggregate quantity and strength results also in an increase in bond strength. More details on the factors that affect the bond of reinforcing steel to concrete can be found in ACI 408R-03.

Bond characteristics of conventional carbon steel reinforcement and epoxy-coated reinforcement with concrete has been thoroughly investigated by many researchers and addressed in terms of bond or development length. Their experimental results contributed to the ACI Committee 408 database on "Bond and Development of Straight Reinforcing Bars in Tension" and were used in formulating the current equations in both ACI 318-02 and ACI 408R-03 to predict the bond force.

The experimental investigation presented in this paper is designed to study the bond behavior of the high-strength steel (commercially known as MMFX steel bar) and included the effect of bar size, level of confinement with transverse reinforcement, bonded length, and bar cast position. The MMFX steel bars exhibit superior mechanical properties when compared with conventional steel reinforcement, and the requirements covering deformation dimensions of ribs (length, height, and frequency along bars) are the same as conventional steel bars and conform to ASTM A 1035-06 and ASTM A 615-04. The validity of such innovative reinforcement and its ability to transfer stresses to the surrounding concrete through bond must be considered. To the best knowledge of the authors, very little information is available about the bond strength of high-strength steel reinforcement. Therefore, MMFX steel reinforcement was considered to provide a database and knowledge of the bond of high-strength steel to concrete and to compare the behavior to conventional Grade 420 MPa (60 ksi) carbon steel reinforcement.

RESEARCH SIGNIFICANCE

Typical design code equations attempt to predict a required bonded length that will result in yielding of the bar before bond failure. The importance of conducting an experimental investigation to identify the bond characteristics...
possible conical failure, the first 102 mm (4 in.) of the MMFX steel bar from the concrete surface at the loaded end were debonded using a plastic tube, as shown in Fig. 2. The test matrix for the beam-end specimens program is shown in Table 1. Details of the test setup are shown in Fig. 3.

The MMFX steel bar was tensioned using a hydraulic jack, and the specimen was held in place using steel beams and high-strength Dywidag steel bars anchored to the laboratory floor, as shown in Fig. 3. The applied tension load was measured using a load cell placed at the jacking side of the beam-end specimen. Three 6 mm (0.25 in.) 120 ohm electrical resistance strain gauges, installed on the bonded surface of each MMFX steel bar, were used to measure the strain distribution along the bonded length of the bar. As the applied load increased, excessive increase in the width of the first flexural crack near the loaded end was observed. Figure 5 shows the tensile splitting mode of bond failure, as shown in Fig. 4. An excessive increase in the width of the first flexural crack near the loaded end was observed. Figure 5 shows the tensile stress and the average bond stress developed in the MMFX steel bar in Beam B4 versus the measured slip at the loaded and unloaded ends of the bar.

Bond distribution—Figure 6 shows the bond stress distribution \( u_{bl} \) at distance \( x \) from the loaded end along the bonded length of the MMFX steel bar for Beam B1 at different stress levels based on the linear behavior of MMFX within the elastic range to determine the stress \( f_{bl} \) corresponding to the measured strain. Figure 6 was developed as follows: at a given bar stress level, the corresponding strain readings along the bonded length, as measured by the three strain gauges, were determined. Using the stress-strain curve obtained from the mechanical properties of the MMFX steel bar, the corresponding stress levels were obtained for each strain gauge reading along the bar. These stress values were
Table 3—Test matrix of Experimental Program 2: beam-splice specimens

<table>
<thead>
<tr>
<th>Group no.</th>
<th>Specimen ID</th>
<th>Beam dimensions</th>
<th>Concrete dimensions</th>
<th>Cover and spacing</th>
<th>Stirrup details along splice length</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>B-6-12</td>
<td>305</td>
<td>4877</td>
<td>305</td>
<td>60.3</td>
</tr>
<tr>
<td>II</td>
<td>B-6-24</td>
<td>610</td>
<td>4877</td>
<td>311</td>
<td>54.0</td>
</tr>
<tr>
<td>III</td>
<td>B-6-36</td>
<td>914</td>
<td>4877</td>
<td>305</td>
<td>54.0</td>
</tr>
<tr>
<td>IV</td>
<td>B-6-60</td>
<td>1524</td>
<td>6096</td>
<td>305</td>
<td>57.2</td>
</tr>
<tr>
<td>I</td>
<td>B-8-12</td>
<td>305</td>
<td>4877</td>
<td>305</td>
<td>123.8</td>
</tr>
<tr>
<td>II</td>
<td>B-8-24</td>
<td>610</td>
<td>4877</td>
<td>305</td>
<td>123.8</td>
</tr>
<tr>
<td>III</td>
<td>B-8-48</td>
<td>1219</td>
<td>4877</td>
<td>311</td>
<td>114.3</td>
</tr>
<tr>
<td>IV</td>
<td>B-8-72</td>
<td>1829</td>
<td>6096</td>
<td>305</td>
<td>120.7</td>
</tr>
</tbody>
</table>

*Dimensions of specimens were measured after casting.

Phase II: beam-splice specimens

A total of eight large-scale concrete beams reinforced with MMFX steel bars spliced at the midspan were tested. Four specimens were each reinforced with two No. 19 (No. 6) MMFX steel bars and the other four with one No. 25 (No. 8) MMFX steel bar. The beam-splice specimens were divided into four groups, as shown in Table 3. Each group consisted of two specimens with identical concrete dimensions but had different amounts (one or two) and sizes (No. 19 or No. 25 [No. 6 or No. 8]) of the reinforcing MMFX steel bars. The specimens in Groups I and II were rectangular in cross section, whereas those in Groups III and IV had T-shaped cross sections. The spliced lengths varied from one specimen to another and ranged from 305 to 1829 mm (1.0 to 6.0 ft), as shown in Table 3. To minimize the effect of the applied loads on the spliced length, the distance between the end of the splice length and the center of the applied load was always more than 305 mm (1.0 ft). The specimens with No. 25 (No. 8) MMFX steel bars, double-legged closed stirrups were evenly distributed along the splice length to provide the required level of confinement around the spliced bars. To prevent possible premature shear failure, shear reinforcement was provided using No. 10 (No. 3) Grade 60 double-legged closed stirrups spaced at 127 mm (5 in.) along the shear span for all tested beam specimens. Compression reinforcement was provided by two No. 13 (two No. 4) Grade 420 MPa (60 ksi) steel bars as top reinforcement. The variation in the beam’s dimensions was selected to achieve different stress levels in the MMFX steel bar length at failure. The bottom and side concrete covers and the transverse spacing between the spliced bars were kept constant for all the beams reinforced with two No. 19 (two No. 6) MMFX steel bars. The selected values were 1.8d_b, 3d_b, and 6d_b, where d_b is the bar diameter. For beams reinforced with one No. 25 (No. 8) MMFX steel bar, the bottom and side concrete covers were also kept constant at values of 1.375d_b and 5d_b, respectively. Table 3 shows the actual measured dimensions of the specimens after casting.

The test matrix for the beam-splice test program is shown in Table 3. The first letter of the beam designation "B" stands for Beam; the middle number identifies the bar size—No. 19 and No. 25 (No. 6 and No. 8); whereas the last number represents the spliced length of the bar, in inches. Standard concrete cylinders, 102 x 204 mm (4 x 8 in.), were cast according to ASTM C 31-00\(^{23}\) for the purpose of determining the compressive strength of the concrete. The concrete cylinders were cured in the same manner as the test specimens. The average concrete compressive strength, determined using three cylinders according to ASTM C 39-01,\(^{24}\) at the age of 28 days was 41.8 MPa (6071 psi). Table 4 shows the concrete compressive strengths as measured on the day of testing.

All beams were simply supported loaded in four-point bending. The load was applied using an MTS actuator operated
just before failure, calculated using the moment curvature analysis, are given in Table 4.

**Evaluation of bond strength and splice length**

The experimental results of the bond force of No. 19 and No. 25 (No. 6 and No. 8) MMFX steel bars were compared with the predictions from the equation proposed by Zuo and Darwin and the ACI 318-02 equation. The bond force was calculated from the bar stress determined using the experimental stress-strain curve of the MMFX bar (Fig. 1) for the corresponding measured strain reading in the bar, as measured by the strain gauges attached to the MMFX steel bars.

**Zuo and Darwin equation**—The bond capacity of the No. 19 and No. 25 (No. 6 and No. 8) MMFX steel bars was evaluated using the equation proposed by Zuo and Darwin, as shown in Fig. 13 and 14, respectively. Test results indicated that the Zuo and Darwin equation provided conservative prediction of the bond capacity of No. 19 (No. 6) MMFX steel bars at stress levels up to the proportional strength of 579 MPa (84 ksi), and provided a very close prediction to the bond capacity of No. 25 (No. 8) MMFX steel bars up to the proportional strength of 607 MPa (88 ksi). At higher stress levels, however, the equation resulted in unconservative prediction for both No. 19 and No. 25 (No. 6 and No. 8) MMFX steel bars. Test results suggested that there is a need to modify the Zuo and Darwin equation to include a higher stress level. The stress limitation imposed by ACI 318-02 for the maximum allowed design yielding strength of 550 MPa (80 ksi) has been selected as an upper boundary for using the equation by Zuo and Darwin. Therefore, modification of the equation has been proposed beyond the stress level of 550 MPa (80 ksi).

Bond equations generally relate bar stresses to bond lengths. The equation proposed by Zuo and Darwin for bond strength of bars not confined with transverse reinforcement ($T_b = T_c$ and $T_s = 0$), in terms of bond force $T_b$, as is the case for Beams B6-12, B6-24, B6-36, and B6-60, is given by

\[
\frac{T_b}{f'_{c, 1/4}} = \frac{T_c}{f'_{c, 1/4}} = \frac{1}{1.43} \left[ 15.2 + 3.45 (c_{\min} + 0.5 d_s) + 56.2 A_s \left( 0.1 \frac{c_{\max}}{c_{\min}} + 0.9 \right) \right] \quad (2a)
\]

Imperial units

\[
\frac{T_b}{f'_{c, 1/4}} = \frac{T_c}{f'_{c, 1/4}} = \frac{1}{59.8} \left[ 15.2 + 3.45 (c_{\min} + 0.5 d_s) + 2350 A_s \left( 0.1 \frac{c_{\max}}{c_{\min}} + 0.9 \right) \right] \quad (2b)
\]

To limit the applicability of Eq. (2a) and (2b) to cases in which a splitting failure governs

\[
\frac{1}{d_b} \left[ (c_{\min} + 0.5 d_s) \left( 0.1 \frac{c_{\max}}{c_{\min}} + 0.9 \right) \right] \leq 4.0 \quad (2c)
\]

Equations (2a) or (2b) were modified by changing only the numerical constants to ensure conservative predictions at any stress level between 550 and 831 MPa (80 and 120.5 ksi). The proposed modification of the Zuo and Darwin equation for MMFX beyond the stress level of 550 MPa (80 ksi) is

\[
\frac{T_b}{f'_{c, 1/4}} = \frac{T_c}{f'_{c, 1/4}} = \frac{1}{5.3} \left[ 15.2 + 3.45 (c_{\min} + 0.5 d_s) + 2350 A_s \left( 0.1 \frac{c_{\max}}{c_{\min}} + 0.9 \right) \right] \quad (3a)
\]
Imperial units

\[
\frac{T_b}{f'_{c}} = \frac{T_c}{f'_{c}} = \frac{10}{3} l_d \pi (c'_{\text{min}} + 0.5d_b) \tag{6b}
\]

Equations (6a) and (6b) can be rewritten in terms of \(l_d\) as follows

SI units

\[
\frac{l_d}{d_b} = \frac{9}{10} \frac{f_y}{f'_{c}} \left( \frac{1}{c'_{\text{min}} + 0.5d_b} \right) \tag{7a}
\]

Imperial units

\[
\frac{l_d}{d_b} = \frac{3}{40} \frac{f_y}{f'_{c}} \left( \frac{1}{c'_{\text{min}} + 0.5d_b} \right) \tag{7b}
\]

Equations (7a) and (7b) were written in terms of the yield stress. To determine the development length of the bar in the tested beams, however, this stress should be taken as the actual measured stress in the MMFX steel bars.

Based on the tested results, Eq. (6a) or (6b) was modified by changing the numerical constants and addition of a term function of \(d_b\). The proposed modification of the ACI 318-02\textsuperscript{13} equation for MMFX beyond stress level of 550 MPa (80 ksi) is

SI units

\[
\frac{T_b}{f'_{c}} = \frac{T_c}{f'_{c}} = \frac{18}{10} l_d \pi (c'_{\text{min}} + 0.5d_b) + 40d^2_b \tag{8a}
\]

Imperial units

\[
\frac{T_b}{f'_{c}} = \frac{T_c}{f'_{c}} = \frac{13}{10} l_d \pi (c'_{\text{min}} + 0.5d_b) + 500d^2_b \tag{8b}
\]

Equations (8a) and (8b) can be rewritten in terms of \(l_d\) as follows

SI units

\[
\frac{l_d}{d_b} = \frac{f_y}{f'_{c}} - \frac{51}{40 (c'_{\text{min}} + 0.5d_b)} \left( \frac{1}{d_b} \right) \tag{9a}
\]

Imperial units

\[
\frac{l_d}{d_b} = \frac{f_y}{f'_{c}} - \frac{637}{52 (c'_{\text{min}} + 0.5d_b)} \left( \frac{1}{d_b} \right) \tag{9b}
\]

Using Eq. (8a) or (8b) to predict the bond capacity of No. 19 (No. 6) MMFX steel bars for stress levels higher than 550 MPa (80 ksi), the maximum test/predict ratio was approximately 1.19, as can be seen in Fig. 15.

The current ACI 318-02\textsuperscript{13} equation, for bars confined with transverse reinforcement \((T_b = T_c + T_b)\), in terms of bond force \(T_b\) as is the case for Beams B8-12, B8-24, B8-48, and B8-72, is given by

SI units

\[
\frac{T_b}{f'_{c}} = \frac{T_c}{f'_{c}} + \frac{T_b}{f'_{c}} = \frac{5}{18} l_d \pi (c'_{\text{min}} + 0.5d_b) + \frac{5}{18} l_d A_{tr} f_{yr} \tag{10a}
\]

Imperial units

\[
\frac{T_b}{f'_{c}} = \frac{T_c}{f'_{c}} + \frac{T_b}{f'_{c}} = \frac{10}{3} l_d \pi (c'_{\text{min}} + 0.5d_b) + \frac{10}{3} l_d A_{tr} f_{yr} \tag{10b}
\]

Equations (10a) and (10b) can be rewritten in terms of \(l_d\) as follows

SI units

\[
\frac{l_d}{d_b} = \frac{9}{10} \frac{f_y}{f'_{c}} \left( \frac{1}{c'_{\text{min}} + 0.5d_b + k_{tr}} \right) \tag{11a}
\]

and \(k_{tr} = A_{tr} f_{yr} / 10.34sn\)

Imperial units

\[
\frac{l_d}{d_b} = \frac{3}{40} \frac{f_y}{f'_{c}} \left( \frac{1}{c'_{\text{min}} + 0.5d_b + k_{tr}} \right) \tag{11b}
\]

and \(k_{tr} = A_{tr} f_{yr} / 1500sn\)
Fig. 19—Steel bar stress versus splice length/bar diameter for beams reinforced with No. 19 (No. 6) and No. 25 (No. 8) MMFX steel bars.

Fig. 20—Load-midspan deflection for beams reinforced with No. 19 (No. 6) and No. 25 (No. 8) MMFX steel bars.

is nearly linearly related, but not proportional, to the splice length to the bar diameter ratio \( L_d/d_b \) up to the minimum yield strength for No. 19 and No. 25 (No. 6 and No. 8) MMFX steel bars. The relationship also suggests that a splice length of \( 30d_b \) can be safely used to achieve the maximum yield strength of 550 MPa (80 ksi) limited by ACI 318-02.\(^\text{13}\) As shown in Fig. 19, a splice length of \( 45d_b \) can be used to achieve the yield strength of 758 MPa (110 ksi) for MMFX steel bars. The linear, not proportional, relationships extend to a stress of 831 MPa (120.5 ksi), which corresponds to a splice length of \( 50d_b \). Beyond the yield strength, the relationship becomes highly nonlinear and significant splice length is required to achieve higher stress levels, which could be impractical to use for typical applications.

Load-midspan deflection—The load-midspan deflection behavior for the beam-splice specimens reinforced with two No. 19 (No. 6) MMFX bars and specimens reinforced with one No. 25 (No. 8) MMFX bar with different splice lengths ranging from 305 to 1829 mm (12 to 72 in.) are shown in Fig. 20. Two important points are worth noting. As can be seen, for the beams reinforced with two No. 19 (No. 6) MMFX bars or with one No. 25 (No. 8) MMFX bar, the behavior is affected by the splice length in which increasing the splice length increased the deflection at ultimate. By comparing the beams with the same splice length (such as B-6-12 and B-8-12) throughout the post-cracked portion of the load-midspan deflection, and at any given load level, the deflection for the beams with a higher reinforcement ratio is always less than that for beams with a lower reinforcement ratio. This behavior is due to the higher stiffness as a result of the higher reinforcement ratio. Throughout the post-cracking behavior, the stiffnesses presented by the slopes of the curves were also different. For beams with a lower reinforcement ratio, the slope was less than for beams with a higher reinforcement ratio. The difference was found to be closely matching the difference in the reinforcement ratio between the beams with higher reinforcement ratios and the beams with lower reinforcement ratios. The ultimate load-carrying capacity is higher for beams with higher reinforcement ratios than beams with lower reinforcement ratios. Note that the results of Beams B-6-36 and B-8-48 are not shown in Fig. 20 due to problems that occurred during the test in measuring the deflection at midspan.

**CONCLUSIONS**

The following conclusions were made based on the limited number of tested specimens:

1. Bond behavior of the MMFX steel bars is similar to that of the conventional Grade 420 MPa (60 ksi) carbon steel up to the stress level corresponding to the proportional limit, imposed by ACI 318-02,\(^\text{13}\) of 550 MPa (80 ksi). At higher stress levels, bond failure changed from the typical sudden and brittle failure, normally observed for conventional steel, to a gradual and ductile failure due to the nonlinear behavior of the MMFX steel bars in this range;

2. The nonlinear ductile response of the MMFX bars at high stress levels beyond proportional limit strength has a strong influence in reducing the bond strength of the MMFX bars compared with Grade 420 MPa (60 ksi) steel;

3. The current equations proposed by Zuo and Darwin\(^\text{12}\) and ACI 318-02\(^\text{13}\) for bond force provided conservative prediction of the bond capacity for No. 19 and No. 25 (No. 6 and No. 8) MMFX steel bars up to 550 MPa (80 ksi). For stress levels exceeding 550 MPa (80 ksi) and up to a stress level of 831 MPa (120.5 ksi) for No. 19 (No. 6) MMFX bars, and 955 MPa (138.5 ksi) for No. 25 (No. 8) MMFX bars, respectively, both equations were modified to provide better prediction of the bond force capacity; and

4. The splice length to bar diameter ratio is nearly linearly, but not proportionally, related to the induced stress in the MMFX steel bar up to yield strength. The relationship becomes highly nonlinear beyond a stress level of 758 MPa (110 ksi).

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**NOTATION**

- \( A_{st} \) = area of each stirrup or tie crossing potential plane of splitting adjacent to reinforcement being developed or spliced
- \( B_f \) = width of flange of beam
- \( b_w \) = width of web of beam
- \( c_b \) = thickness of clear bottom concrete cover
- \( c_{max} \) = maximum of \( c_s \) or \( c_l \)
- \( c_{min} \) = minimum of concrete covers surrounding bar or half clear spacing between bars, minimum of \( c_{1s} \) and \( c_s \)
- \( c_{min} \) = minimum of \( c_s \) or \( c_{1s} \)
- \( c_s \) = minimum of \( c_{1s} \) or \( c_{2s} + 6.35 \text{ mm} (c_{1s} + 0.25 \text{ in.}) \)
- \( c_l \) = half clear spacing between spliced bars
- \( c_{1lo} \) = thickness of clear side concrete cover

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