EVALUATION OF BOND CHARACTERISTICS
OF MMFX STEEL

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1. Summary

This report summarizes the research findings related to the bond characteristics of the commercially available high-strength steel known as Micro-composite Multi-structural Formable (MMFX) steel. The results are based on the experimental program conducted at North Carolina State University (NCSU) as part of a coordinated research program in collaboration with the University Kansas (KU) and the University of Texas at Austin (UT). The experimental program at each university comprised of testing twenty-two large scale beam-splice specimens.

For the twenty-two beams tested at NCSU, splitting was the prevailing mode of failure except for five beams that failed in flexure. The use of large than intended amount of transverse reinforcement to confine the spliced bars in these five beams resulted in significant increase in bond strength and allowed the beams to fail in flexural mode. Failure of beams with unconfined spliced bars was sudden in an abrupt manner. Use of transverse reinforcement to confine the spliced bars caused more gradual failure accompanied with visible splitting cracks prior to failure.

Test results indicated that a stress level of 90 and 70 ksi can be developed in No. 8 and No. 11 MMFX spliced bars, respectively, without the use of transverse reinforcement. Confining the MMFX spliced bars by transverse reinforcement allowed the No. 8 and No. 11 MMFX bars to develop a stress level of 150 ksi. Use of transverse reinforcement increased the ultimate load capacity and ductility of the tested beams. Therefore, whenever possible it is recommended to confine the MMFX spliced bars by transverse reinforcement to utilize their high strength and to ensure adequate level of ductility.

Based on a statistical evaluation of the test data and the average of the measured versus prediction ratio to assess the current bond equations, it was found that ACI 318-05 code equation overestimates the strength of unconfined spliced MMFX bars, especially for high strength concrete. The bond equation proposed by ACI committee 408 provides a better estimate of the stresses for unconfined spliced bars and exhibits less scatter of data in comparison with the ACI 318-05 equation. For a more accurate estimation of the stresses in unconfined spliced MMFX bars, the following simple equation is proposed, based on this research program:
The statistical evaluation of the test data and the average of the measured versus prediction ratio showed that the ACI 318-05 and ACI committee 408 equations can be safely used to estimate the bond strength of spliced MMFX bars confined by transverse reinforcement.

\[ f_s = \frac{1100^4 f_c \sqrt{f_s} \sqrt{c}}{d_b} \text{ (psi)} \]
2. Introduction

Splicing of reinforcing steel bars is a common practice in concrete structures. Within the splice length the tensile forces are transferred from one bar to the other through the surrounding concrete. If sufficient bond is developed between the concrete and the reinforcing steel bars, both materials act in a composite action to resist the applied load.

The commercially available Micro-composite Multi-structural Formable (MMFX) steel is characterized by its high tensile strength, non-linear behavior above stress level of 120 ksi and lack of well-defined yield plateau. For MMFX steel to be used as main reinforcement for concrete structures and to utilize its high tensile strength, a full understanding of its bond behavior to the concrete is of paramount importance. The current knowledge of bond strength is primarily empirical, as do the descriptive design equations and code provisions. The equations were empirically derived based on research conducted using steel with a yield strength limited to 80 ksi. Therefore, for MMFX steel to be effectively and safely utilized, current code provisions need to be examined comprehensively.

This study was undertaken to investigate the bond characteristics of MMFX steel reinforcing bars and to evaluate the applicability of current code equations. The research at North Carolina State University (NCSU) is a part of a coordinated research program in collaboration with the University of Kansas (KU) and the University of Texas at Austin (UT). Detailed test data collected from the experimental program at NCSU are given in the Appendix of this report.
3. Test Specimens

Large-scale beam-splice specimens were used to study the bond characteristics of MMFX steel reinforcing bars to concrete. Beam-splice specimen is recommended by ACI Committee 408 since it provides the most realistic state of stresses in comparison to other test configurations. In beam-splice specimen the reinforcing bar is subjected to tensile stresses, while the surrounding concrete is subjected to localized compressive forces at the contact bearing areas due to the relative displacement of the bar with respect to the concrete. Based on the consensus of the investigators participating in this study, the test beams were selected to have equal side and top concrete covers, as well as clear bars spacing equal to twice the selected concrete cover.

The collective experimental program was designed to include the following selected important parameters affecting the bond strength:

- **Bar size:** No. 5, 8, and 11
- **Target Concrete Compressive Strength:** 5000 and 8000 psi
- **Concrete Cover:**
  - ¾ in., 2\(d_b\), and 3\(d_b\) for No. 5 bars
  - 1.5 and 2.5 in. for No. 8 bars
  - 2.0 and 3.0 in. for No. 11 bars
- **Splice Length:**
  - Two splice lengths to achieve bar stress of 80 and 100 ksi without the use of transverse reinforcement
- **Confinement Level:**
  - First level is to achieve 100 and 120 ksi for shorter splice length
  - Second level is to achieve 120 and 140 ksi for longer splice length

The collective test matrix for the three universities is given in Table 1. According to the collective test matrix, the experimental program at each university comprised of twenty-two specimens. It should be noted that the test matrix includes twelve duplicate beams for providing cross checks among the three universities. These common beams are highlighted in Table 1. The design of the splice lengths to achieve the selected
stresses in the bars was calculated according to the bond equation proposed by ACI Committee 408\(^1\) (equation 4-11a, ACI 408R-03). Similarly, the amount of transverse reinforcement required to achieve the selected stresses in the spliced bars was determined according to the same equation. It should be noted that the strength-reduction factor (Φ-factor) was taken equal to unity in all calculations.

Table 1: Collective test matrix for the three universities

<table>
<thead>
<tr>
<th>f(_c') psi</th>
<th>Bar Size</th>
<th>Kansas University (KU)</th>
<th>North Carolina State University (NCSU)</th>
<th>University of Texas at Austin (UT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Cover</td>
<td>Cover</td>
<td>Cover</td>
<td>Cover</td>
</tr>
<tr>
<td></td>
<td>¾ in.</td>
<td>2(_d_b)</td>
<td>3(_d_b)</td>
<td>¾ in.</td>
</tr>
<tr>
<td></td>
<td>OC0</td>
<td>OC0</td>
<td></td>
<td>OC0</td>
</tr>
<tr>
<td></td>
<td>XC0</td>
<td>XC0</td>
<td></td>
<td>XC0</td>
</tr>
<tr>
<td>5000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
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<td>Cover</td>
<td>Cover</td>
<td>Cover</td>
<td>Cover</td>
</tr>
<tr>
<td></td>
<td>1.5 in.</td>
<td>2.5 in.</td>
<td>1.5 in.</td>
<td>2.5 in.</td>
</tr>
<tr>
<td></td>
<td>OC0,1,2</td>
<td>OC0,1,2</td>
<td>OC0,1,2</td>
<td>OC0,1,2</td>
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<td>XC0,1,2</td>
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</tr>
<tr>
<td>11</td>
<td>Cover</td>
<td>Cover</td>
<td>Cover</td>
<td>Cover</td>
</tr>
<tr>
<td></td>
<td>2.0 in.</td>
<td>3.0 in.</td>
<td>2.0 in.</td>
<td>3.0 in.</td>
</tr>
<tr>
<td></td>
<td>OC0,1,2</td>
<td></td>
<td>OC0,1,2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>XC0,1,2</td>
<td></td>
<td>XC0,1,2</td>
<td></td>
</tr>
<tr>
<td>8000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Cover</td>
<td>Cover</td>
<td>Cover</td>
<td>Cover</td>
</tr>
<tr>
<td></td>
<td>1.5 in.</td>
<td>2.5 in.</td>
<td>1.5 in.</td>
<td>2.5 in.</td>
</tr>
<tr>
<td></td>
<td>OC0,1,2</td>
<td>OC0,2</td>
<td>OC0,1,2</td>
<td></td>
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<td>Cover</td>
<td>Cover</td>
<td>Cover</td>
<td>Cover</td>
</tr>
<tr>
<td></td>
<td>2.0 in.</td>
<td>3.0 in.</td>
<td>2.0 in.</td>
<td>3.0 in.</td>
</tr>
<tr>
<td></td>
<td>OC0,1,2</td>
<td></td>
<td>OC0,1,2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>XC0,1,2</td>
<td></td>
<td>XC0,1,2</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>22</td>
<td>22</td>
<td>22</td>
<td></td>
</tr>
</tbody>
</table>
A four-part notation system was developed to identify the tested beams. The notation of the beams used in Table 1 and hereafter is as follows: the first letter “8 or 11” designates the size of spliced bars and the second letter “5 or 8” designates the targeted concrete strength in ksi. The third letter “O or X” designates the selected splice length to achieve a specified stress level of 80 or 100 ksi without confinement, respectively. The last two letters “C0, C1 or C2” designates the unconfined and the two selected confinement levels in the splice zone, respectively.

The twenty-two beams to be tested at NCSU were divided into eight groups as shown in Figure 1. The beams within each group are identical in all aspects except for the amount of confining transverse reinforcement along the splice length.

![Experimental Program, 22 Beams](image)

**Figure 1: Experimental program at NCSU**
4. Experimental Program

4.1. Design of Test Beams

Design of the tested beams was based on cracked-section analysis employing modified Hognestad stress-strain model of the concrete, as shown in Figure 2. It should be noted the cracked-section analysis neglects the tensile strength of the concrete. The equations used in the cracked-section analysis relating stress to strain of MMFX steel and describing Hognestad stress-strain relationship of concrete are as follows:

For MMFX steel:

\[ f_s = 177 \left( 1 - e^{-185 \varepsilon_s} \right) \]

For concrete

\[ E_c = 1.8E6 + 460 f_c' \]

\[ \varepsilon_o = 1.7 f_c' / E_c \]

For the 2nd degree parabola

\[ f_c = f_c'' \left[ \frac{2 \varepsilon_c}{\varepsilon_o} - \left( \frac{\varepsilon_c}{\varepsilon_o} \right)^2 \right] \]

(a) Hognestad stress-strain relationship  \hspace{1cm} (b) Section analysis of test beams

Figure 2: Hognestad stress-strain relationship and sectional analysis of test beams

In order to control the depths of the cross-sections, two No. 6 or No. 11 MMFX bars were used as compression steel for beams reinforced with No. 8 or No. 11 spliced bars, respectively. The cross-sectional dimensions and nominal strengths resulting from the flexural design for all the beams tested at NCSU are given in Table 2. It should be
noted that the width of the beams was dictated by the spliced bar diameter and the specified concrete cover.

Table 2: Design of NCSU test beams

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Bar Size</th>
<th>$f_c$ psi</th>
<th>Cover in.</th>
<th>Section “b x h” in.</th>
<th>Stresses in Steel ksi</th>
<th>Moment Capacity kips-ft</th>
<th>Applied Load kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8-5-O-C0</td>
<td>5000</td>
<td>2.5</td>
<td>14 x 24</td>
<td>162</td>
<td>40</td>
<td>400</td>
<td>67</td>
</tr>
<tr>
<td>1</td>
<td>8-5-O-C1</td>
<td>5000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
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<td>8-5-O-C2</td>
<td>5000</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>2</td>
<td>8-5-X-C0</td>
<td>8</td>
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<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>8-5-X-C1</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>8-5-X-C2</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>8-8-O-C0</td>
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<td>10 x 24</td>
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<td>445</td>
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<tr>
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<td></td>
</tr>
<tr>
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<td>11</td>
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<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>11-8-O-C0</td>
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<td>3.0</td>
<td>18 x 24</td>
<td>161</td>
<td>23</td>
<td>754</td>
<td>126</td>
</tr>
<tr>
<td>7</td>
<td>11-8-O-C1</td>
<td>8000</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>11-8-O-C2</td>
<td>8000</td>
<td></td>
<td></td>
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<td></td>
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</tr>
<tr>
<td>8</td>
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<td>18 x 24</td>
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<td>8</td>
<td>11-8-X-C1</td>
<td>8000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>11-8-X-C2</td>
<td>8000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The spacing of the stirrups provided in the shear spans were designed to prevent premature shear failure. All the stirrups used were No. 4 Grade 60 steel. Reinforcement details of the test beams are shown in Figure 3 and given in Table 3.

Each group of beams was cast at the same time using the same batch of concrete. Normal weight concrete with maximum aggregate size of 3/8 in. was used for constructing all test beams. The beams were cast with the spliced bars in the bottom position to avoid the reduction of the bond strength for top-cast bars. However, before testing the beams were rotated 180 degrees about its longitudinal axis to have the spliced bars in the top position to facilitate mapping the cracks and test observations. Therefore, two plastic tubes were inserted at each end of the beam before casting to enable rotating the beams with the use of two steel rods inserted into the tubes and the hooks of the overhead crane. The ten beams reinforced with No. 8 bars before testing are shown in Figure 4.
## Table 3: Reinforcement details of NCSU test beams

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Bar Size</th>
<th>Section &quot;b x h&quot; in.</th>
<th>Cover in.</th>
<th>Splice length in.</th>
<th>Comp. Steel</th>
<th>Stirrups Spacing in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8-5-O-C0</td>
<td>8-5-O-C1</td>
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<td></td>
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<tr>
<td></td>
<td>1</td>
<td></td>
<td>14 x 24</td>
<td>2.5</td>
<td>31</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>8-5-X-C0</td>
<td>8-5-X-C1</td>
<td>8-5-X-C2</td>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td></td>
<td>14 x 24</td>
<td>2.5</td>
<td>31</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>8-8-O-C0</td>
<td>8-8-O-C1</td>
<td>8-8-X-C0</td>
<td>4.0</td>
<td>10.5</td>
<td></td>
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<td>3</td>
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<td>10 x 24</td>
<td>1.5</td>
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</tr>
<tr>
<td>4</td>
<td>8-8-X-C0</td>
<td>8-8-X-C1</td>
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</tr>
<tr>
<td></td>
<td>4</td>
<td></td>
<td>18 x 24</td>
<td>3.0</td>
<td>57</td>
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</tr>
<tr>
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<td></td>
</tr>
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<td>6</td>
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<td>11-5-X-C1</td>
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<td>14 x 36</td>
<td>2.0</td>
<td>69</td>
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<td></td>
</tr>
<tr>
<td>7</td>
<td>11-8-O-C0</td>
<td>11-8-O-C1</td>
<td>11-8-O-C2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>7</td>
<td></td>
<td>18 x 24</td>
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<td>43</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>11-8-X-C0</td>
<td>11-8-X-C1</td>
<td>11-8-X-C2</td>
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<td></td>
<td>18 x 24</td>
<td>3.0</td>
<td>57</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* See Figure 3
4.2. Test Setup and Instrumentation

All beams were tested in four-points bending configuration to develop a constant moment zone where the spliced bars were located. The load was applied using hydraulic jacks reacting against the strong floor and the test beams were supported by tying down strong steel beams to the floor using prestressing bars. Load cells were placed between the test beam and the supporting steel beams to measure the reactions at the supports. Schematic diagram and a picture of the test setup used for testing at NCSU are shown in Figure 5. The beams were tested with the spliced bars in the top position to facilitate mapping the cracks and test observations.
The load was applied using two hydraulic jacks at each location with a capacity of 120 kips each. Two 150 kips load cells were used to measure the reaction at the supports. Four electrical resistance strain gages were attached to the spliced bars before casting the concrete. The strain gages were located immediately outside the splice zone to measure the strain in the spliced bars. Six string pots were used to measure the deflections at the mid-span, at the location of the load, and at the support. Six PI gages with gage length of 100 mm were mounted to the top and bottom surfaces of the beam in the longitudinal direction to record the concrete strain including the crack width within the gage length during loading. The PI gages were located at both ends of the splice as well as mid-span. In addition, a crack comparator was used to manually measure the crack width at different load levels. A data acquisition system was used to electronically record the data.
5. Test Results of NCSU Test Beams

5.1. General

The beams tested at NCSU were divided into eight groups according to the specified concrete cover, target concrete strength, and the splice length. The beams within each group are identical in all aspects except for the amount of confining transverse reinforcement along the splice length. The bond characteristics of MMFX steel reinforcing bars will be evaluated in light of the mode of failure of beams, developed stresses in the spliced bars, load-deflection behavior, and crack pattern. It should be noted that only typical test results will be given in the following sections, while the detailed test results are given in the Appendix. Based on the measured stress-strain relationship by UT, the following exponential equations for modeling MMFX steel bars were used in all subsequent computations even though a slightly different equation was used for the initial design of the test beams:

\[ f_s = 156 \left(1 - e^{-220\varepsilon_s}\right) \quad \text{for No. 8 bars} \]

\[ f_s = 162 \left(1 - e^{-235\varepsilon_s}\right) \quad \text{for No. 11 bars} \]

The deformation characteristics of No. 8 MMFX steel bars used in this study were measured and the bars had parallel deformation pattern (bamboo) as shown in Figure 6. Based on the measured characteristics of the deformation pattern, the relative rib area \(R_r\) of the No. 8 bars used in this study was determined to be 0.092. The measured deformation dimensions conform to ASTM A 615-06 and ASTM A 1035-07. The relative rib area in U.S. practice (ACI 408R-03) is the ratio of the bearing area of the ribs to the shearing area between the ribs, which can be expressed as follows:

\[ R_r = \frac{\text{Projected rib area normal to the bar axis}}{\text{Nominal bar perimeter} \times \text{center-to-center spacing of ribs}} \]
5.2. Mode of Failure

In general, failure due to splitting of the concrete cover was the prevailing mode of failure. However, five beams containing spliced bars confined by transverse reinforcement failed due to flexure as indicated by crushing of the concrete in the compression zone. The use of excess amount of transverse reinforcement to confine the spliced bars in these five beams resulted in an increase in bond force, and thus enabling flexural failure to occur. Beams with spliced bars not confined by transverse reinforcement failed very suddenly in an explosive and abrupt manner as shown in Figure 7. Use of transverse reinforcement to confine the spliced bars caused more gradual failure accompanied with fully visible splitting cracks in the concrete cover giving advance warning. Table 4 gives the mode of failure, and the failure load and the corresponding stresses in the spliced bars for all test beams. In addition, Table 4 also gives the measured concrete compressive strength on the day of testing.
Table 4: Summary of the mode of failure for test beams

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>$f_{c'}$ psi</th>
<th>Failure Mode</th>
<th>Failure Load kips</th>
<th>Stresses in Spliced bars at Failure ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8-5-O-C0</td>
<td>6015</td>
<td>Splitting</td>
<td>40.0</td>
<td>96</td>
</tr>
<tr>
<td></td>
<td>8-5-O-C1</td>
<td></td>
<td>Splitting</td>
<td>58.6</td>
<td>140</td>
</tr>
<tr>
<td></td>
<td>8-5-O-C2</td>
<td></td>
<td>Flexure</td>
<td>65.1</td>
<td>152</td>
</tr>
<tr>
<td>2</td>
<td>8-5-X-C0</td>
<td>5817</td>
<td>Splitting</td>
<td>45.8</td>
<td>110</td>
</tr>
<tr>
<td></td>
<td>8-5-X-C1</td>
<td></td>
<td>Flexure</td>
<td>64.1</td>
<td>152</td>
</tr>
<tr>
<td></td>
<td>8-5-X-C2</td>
<td></td>
<td>Flexure</td>
<td>63.9</td>
<td>152</td>
</tr>
<tr>
<td>3</td>
<td>8-8-O-C0</td>
<td>8400</td>
<td>Splitting</td>
<td>39.9</td>
<td>91</td>
</tr>
<tr>
<td></td>
<td>8-8-O-C1</td>
<td></td>
<td>Splitting</td>
<td>67.1</td>
<td>151</td>
</tr>
<tr>
<td>4</td>
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<td>10200</td>
<td>Splitting</td>
<td>48.0</td>
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<tr>
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<td>8-8-X-C1</td>
<td></td>
<td>Splitting</td>
<td>67.6</td>
<td>151</td>
</tr>
<tr>
<td>5</td>
<td>11-5-O-C0</td>
<td>5344</td>
<td>Splitting</td>
<td>95.4</td>
<td>74</td>
</tr>
<tr>
<td></td>
<td>11-5-O-C1</td>
<td></td>
<td>Splitting</td>
<td>172.9</td>
<td>132</td>
</tr>
<tr>
<td></td>
<td>11-5-O-C2</td>
<td></td>
<td>Splitting</td>
<td>199.0</td>
<td>151</td>
</tr>
<tr>
<td>6</td>
<td>11-5-X-C0</td>
<td>4058</td>
<td>Splitting</td>
<td>93.2</td>
<td>72</td>
</tr>
<tr>
<td></td>
<td>11-5-X-C1</td>
<td></td>
<td>Splitting</td>
<td>165.3</td>
<td>127</td>
</tr>
<tr>
<td></td>
<td>11-5-X-C2</td>
<td></td>
<td>Splitting</td>
<td>202.5</td>
<td>155</td>
</tr>
<tr>
<td>7</td>
<td>11-8-O-C0</td>
<td>6070</td>
<td>Splitting</td>
<td>59.6</td>
<td>78</td>
</tr>
<tr>
<td></td>
<td>11-8-O-C1</td>
<td></td>
<td>Splitting</td>
<td>89.7</td>
<td>116</td>
</tr>
<tr>
<td></td>
<td>11-8-O-C2</td>
<td></td>
<td>Flexure</td>
<td>109.5</td>
<td>152</td>
</tr>
<tr>
<td>8</td>
<td>11-8-X-C0</td>
<td>8383</td>
<td>Splitting</td>
<td>74.4</td>
<td>96</td>
</tr>
<tr>
<td></td>
<td>11-8-X-C1</td>
<td></td>
<td>Splitting</td>
<td>100.4</td>
<td>128</td>
</tr>
<tr>
<td></td>
<td>11-8-X-C2</td>
<td></td>
<td>Flexure</td>
<td>113.0</td>
<td>157</td>
</tr>
</tbody>
</table>
The eight beams containing spliced bars not confined by transverse reinforcement (8-5-O-C0, 8-5-X-C0, 8-8-O-C0, 8-8-X-C0, 11-5-O-C0, 11-5-X-C0, 11-8-O-C0, and 11-8-X-C0) failed due to splitting of the concrete cover. A typical failure of beams with unconfined No. 8 and No. 11 spliced bars are shown in Figures 8 and 9, respectively. Failure of the beams with unconfined spliced bars was very explosive with very little warning associated with spalling and scattering of the concrete cover over the entire splice length. The beams failed very shortly after the initiation of the splitting cracks with sudden loss of the load-carrying capacity.
Figure 8: Beam 8-5-O-C0 at the conclusion of the test

Figure 9: Beam 11-8-O-C0 at the conclusion of the test
The eight beams containing spliced bars with first level of confinement (8-5-O-C1, 8-5-X-C1, 8-8-O-C1, 8-8-X-C1, 11-5-O-C1, 11-5-X-C1, 11-8-O-C1, and 11-8-X-C1) failed due to splitting except for the beam (8-5-X-C1) of the second group which failed due to flexure. Typical beams with No. 8 and No. 11 spliced bars at the conclusion of the test are shown in Figures 10 and 11, respectively. The use of transverse reinforcement to confine the spliced bars increased the ultimate load-carrying capacity of the beams. Failure was brittle but in less violent manner than the beams with unconfined spliced bars. The confining stirrups limited the progress of the splitting crack and enabled the beam to deform more with more flexural cracks until failure occurred due to loss of the concrete cover. Presence of the transverse reinforcement prevented spalling of the concrete cover over the entire splice length. In addition, the transverse reinforcement allowed the widths of the splitting cracks to increase and become fully visible before failure, thus providing adequate warning.

Figure 10: Beam 8-5-O-C1 at the conclusion of the test
The failure of the six beams containing spliced bars with second level of confinement (8-5-O-C2, 8-5-X-C2, 11-5-O-C2, 11-5-X-C2, 11-8-O-C2, and 11-8-X-C2) was mainly due to flexure as indicated by crushing of concrete in the compression zone, except for the two beams of the fifth and sixth groups (11-5-O-C2 and 11-5-X-C2). Crushing of concrete occurred next the location of load application where the bending moment and shearing force reach a maximum value as shown in Figures 12 and 13 for beams with No. 8 and No. 11 bars, respectively. Despite the initiation of the splitting cracks, the transverse reinforcement limited the progression of the splitting crack and eventually changing the mode of failure from splitting failure to flexural failure.
Figure 12: Beam 8-5-O-C2 at the conclusion of the test

Figure 13: Beam 11-8-O-C2 at the conclusion of the test
5.3. **Measured Stresses in Spliced Bars**

The measured stresses in the spliced bars were determined by two methods. The first method used the strains measured by the strain gages attached to the bars at the two ends of the splice zone and the aforementioned exponential equations relating the stresses to the strains of MMFX steel bars. The second method determined the stresses in the spliced bars from cracked-section analysis of the tested beams using the measured applied load and the aforementioned exponential equation for MMFX steel. The cracked-section analysis employs Hognestad\(^2\) stress-strain relationship for concrete as discussed in section 4.1. The measured stresses in the spliced bars using both methods are compared in Table 5. In addition, the measured concrete compressive strength on the day of testing is also given in Table 5.

It is readily seen from Table 5 that confining the spliced bars by transverse reinforcement increased the stresses developed in the bars. It is evident that using of transverse reinforcement to confine the spliced bars limits the progress of the splitting cracks, and thus increases the bond force required to cause splitting failure. The increase in the bond force is translated into increase in the stresses developed in the spliced bars.

The measured stresses indicated that a stress level of 90 and 70 ksi can be developed by No. 8 and No. 11 MMFX spliced bars, respectively without the use of transverse reinforcement. Confining the MMFX spliced bars by transverse reinforcement increased the stresses achieved by No. 8 and No. 11 bars, reaching a stress level of 150 ksi for both bar sizes.

For No. 11 MMFX bars, a splice length of 65 bar diameter did not enhance the stresses developed in the bars, indicating that using long splice lengths without confinement is inefficient way to achieve high stress levels. Therefore, it is recommended to use shorter splice lengths with confinement by transverse reinforcement rather than long unconfined splices.
## Table 5: Measured stresses in the spliced bars

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Bar Size</th>
<th>Cover in.</th>
<th>f’c psi</th>
<th>Splice Length in.</th>
<th>Failure Mode</th>
<th>Strain Gages</th>
<th>Section Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8-5-O-C0</td>
<td>8</td>
<td>2.5</td>
<td>6015</td>
<td>31</td>
<td>Splitting</td>
<td>93</td>
<td>96</td>
</tr>
<tr>
<td></td>
<td>8-5-O-C1</td>
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<td></td>
<td></td>
<td></td>
<td>Splitting</td>
<td>138</td>
<td>140</td>
</tr>
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<td>5817</td>
<td>41</td>
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<td>152</td>
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<td>Flexure</td>
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<td>152</td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td>Flexure</td>
<td>&gt;138</td>
<td>152</td>
</tr>
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<td>92</td>
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</tr>
<tr>
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<td>11-8-X-C2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Flexure</td>
<td>&gt;131</td>
<td>157</td>
</tr>
</tbody>
</table>
5.4. **Load-Deflection Behavior**

The load-deflection behavior of the tested beams reflects the effect of the splice strength on the ultimate load and deformation capacity of the beam. The load-deflection behavior of each group of beams is given in the same figure to emphasize the effect of confinement by transverse reinforcement. The plotted deflection is the total deflection at mid-span of the beam with respect to the deflected position of the ends of the beam. A typical load-deflection behavior of the test beams with No. 8 and No. 11 spliced bars is shown in Figures 14 and 15 for beams of the first and the seventh groups, respectively.

![Graph showing load-deflection behavior](image)

**Figure 14:** Load-deflection behavior of beams of the first group (8-5-O-C0, C1, and C2)
Figure 15: Load-deflection behavior of beams of the seventh group (11-8-O-C0, C1, and C2)

It is clear from the load-deflection behavior that confining the spliced bars by transverse reinforcement increased the ultimate load and deformation capacity of the beams. Beams with spliced bars confined by stirrups exhibited more ductile behavior, with a slow drop in load after the peak. Moreover, the increase in the ultimate load and deflection was governed by the amount of transverse reinforcement used to confine the spliced bars. The beam containing spliced bars not confined by transverse reinforcement failed in a very brittle manner at much lower load and significantly less deflection than the beams with confined spliced bars. In addition, the beam with first level of confinement exhibited markedly less deflection and slightly less load at failure in comparison to the beam with second level of confinement.

The increase in the ultimate load can be directly attributed to the higher bond stresses achieved by the spliced bars, and thus increasing the bond force required to cause failure. In addition, the presence of the transverse reinforcement enabled the spliced bars to achieve very high tensile strains required to achieve the high stresses while preventing the concrete cover from splitting. The high tensile strains in the spliced
bars are translated into increased curvatures at the section level, which in turn is exhibited as higher deformation on the member level. Therefore, it can be concluded that minimum amount of confinement by transverse reinforcement is required to ensure ductile behavior of concrete members.

5.5. Crack Pattern

For all test beams, the first vertical flexural cracks were observed outside the splice zone at or near the location of the applied load (location of maximum moment and shear). In addition, flexural cracks were formed at both ends of the splice length before they were observed inside the splice zone. Flexural cracks propagated downward, and increased in number and in width as the load was increased. Further increase in the load led to the formation of splitting cracks that occurred parallel to the reinforcing bars. The splitting cracks formed initially on the top surface of the beam followed by splitting cracks on the side of the beam at the level of the splices, terminating at the ends of the splice. However, the formation of the splitting cracks did not prohibit the flexural cracks form spreading and propagating towards the compression zone until failure occurs.

When the splitting crack became visible, the crack was marked and the load was recorded as the load at the initiation of the splitting crack. Typical initial splitting crack on the top surface of the beam 8-5-X-C2 is shown in Figure 16. Table 6 gives the measured applied load and the corresponding stresses in the spliced bars at the initiation of the splitting crack, and the width of the corresponding flexural crack. The stresses in the spliced bars at the initiation of the splitting crack were obtained from the cracked-section analysis using the measured applied load. A crack comparator was used to measure the width of the flexural crack and splitting crack at the initiation. When the splitting crack became visible its width was approximately 0.0059 in.
Figure 16: Initiation of splitting crack on the top surface of beam 8-5-X-C2 at 30 kips

It is readily seen from Table 6 that the splitting cracks were initiated when the stresses in the spliced bars reached a range of 70 to 74 ksi and 30 to 50 ksi for beams reinforced with No. 8 and No. 11, respectively. Therefore, it can be justified that the presence of the transverse reinforcement confining the spliced bars did not delay the initiation of the splitting crack; however it limited the progress of the splitting cracks. This finding is confirmed by test observation, as indicated by the continuing progress of the splitting cracks for the confined beams, while unconfined beams failed very shortly after the initiation of the crack. In addition, the average stresses in the No. 8 and No. 11 (≈ 70 and 40, respectively) spliced bars at the initiation of the splitting crack is approximately equal to the inverse ratio of the areas of the bars. This implies that the tension forces in the spliced bars rather than the stresses control the splitting failure. When the tensile force in the spliced bar reaches the capacity of the surrounding concrete, splitting crack initiates.
Table 6: Load and stresses in the spliced bars at initiation of splitting crack

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Cover in.</th>
<th>f'c psi</th>
<th>Splice Length in.</th>
<th>Initiation of Splitting Crack</th>
<th>Flexural Crack Width in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8-5-O-C0</td>
<td>30</td>
<td>73</td>
<td>0.0394</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>8-5-O-C1</td>
<td>30</td>
<td>73</td>
<td>0.0236</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>8-5-O-C2</td>
<td>40</td>
<td>97</td>
<td>0.0492</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>8-5-X-C0</td>
<td>60</td>
<td>31</td>
<td>0.0295</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>8-5-X-C1</td>
<td>58</td>
<td>41</td>
<td>0.0295</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>8-5-X-C2</td>
<td>52</td>
<td>69</td>
<td>0.0197</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>8-8-O-C0</td>
<td>39</td>
<td>32</td>
<td>0.0197</td>
<td>11-5-O-C0</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>8-8-O-C1</td>
<td>40</td>
<td>36</td>
<td>0.0197</td>
<td>11-5-O-C1</td>
<td>0.0157</td>
</tr>
<tr>
<td>4</td>
<td>8-8-X-C0</td>
<td>43</td>
<td>57</td>
<td>0.0197</td>
<td>11-8-X-C0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>8-8-X-C1</td>
<td>44</td>
<td>57</td>
<td>0.0197</td>
<td>11-8-X-C1</td>
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</tr>
<tr>
<td>5</td>
<td>11-5-O-C0</td>
<td>39</td>
<td>32</td>
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<td>11-5-O-C0</td>
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<tr>
<td></td>
<td>11-5-O-C2</td>
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<td>55</td>
<td>0.0157</td>
<td>11-5-O-C2</td>
<td>0.0157</td>
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<tr>
<td>6</td>
<td>11-5-X-C0</td>
<td>38</td>
<td>31</td>
<td>0.0098</td>
<td>11-5-X-C0</td>
<td>0.0098</td>
</tr>
<tr>
<td></td>
<td>11-5-X-C1</td>
<td>38</td>
<td>31</td>
<td>0.0079</td>
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</tr>
<tr>
<td></td>
<td>11-5-X-C2</td>
<td>38</td>
<td>31</td>
<td>0.0098</td>
<td>11-5-X-C2</td>
<td>0.0098</td>
</tr>
<tr>
<td>7</td>
<td>11-8-O-C0</td>
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<td>51</td>
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</tr>
<tr>
<td></td>
<td>11-8-O-C1</td>
<td>40</td>
<td>51</td>
<td>0.0157</td>
<td>11-8-O-C1</td>
<td>0.0157</td>
</tr>
<tr>
<td></td>
<td>11-8-O-C2</td>
<td>40</td>
<td>51</td>
<td>0.0197</td>
<td>11-8-O-C2</td>
<td>0.0197</td>
</tr>
<tr>
<td>8</td>
<td>11-8-X-C0</td>
<td>43</td>
<td>57</td>
<td>0.0295</td>
<td>11-8-X-C0</td>
<td>0.0295</td>
</tr>
<tr>
<td></td>
<td>11-8-X-C1</td>
<td>44</td>
<td>57</td>
<td>0.0295</td>
<td>11-8-X-C1</td>
<td>0.0295</td>
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<tr>
<td></td>
<td>11-8-X-C2</td>
<td>44</td>
<td>57</td>
<td>0.0197</td>
<td>11-8-X-C2</td>
<td>0.0197</td>
</tr>
</tbody>
</table>
The width of the splitting cracks on the top surface of the beams was measured at different load levels using the crack comparator. Typical measured splitting crack width versus stresses in the spliced bars is shown in Figure 17 for an unconfined beam and beam with higher level of confinement. The two beams are shown together to demonstrate the effect of confinement on splitting crack width.

![Figure 17: Splitting crack width of beams of the second group (8-5-X-C0 and C2)](image)

Figure 17 clearly indicates that the splitting crack was initiated at the same stress level for the beams with confined and unconfined spliced bars. The behavior in general indicates that the width of the splitting crack increased with increasing stresses in the spliced bars (increasing of applied load) until failure occurred. However, confining the spliced bars by transverse reinforcement reduced the width of the splitting crack at the same stress level in comparison to unconfined bars. The presence of transverse reinforcement to confine the spliced bars controlled the progress of the splitting cracks until they became fully visible prior to failure giving sufficient warning as shown in Figure 18.
Flexural cracks widths at both ends of the splice were measured at different load levels using PI gages with gage length of 3.94 in. (100 mm). For verification of the PI gages measurements, a crack comparator was also used to measure the crack width at different load levels. A typical flexural crack width versus measured stresses in the spliced bars is shown in Figures 19 and 20 for beams 8-5-O-C2 and 11-8-X-C2 reinforced with No. 8 and No. 11 MMFX spliced bars, respectively. Since, the three beams of the same group are identical in all aspects except for the amount of confinement provided. Therefore, the behavior of each beam was considered representative of the other two beams of the same group. The failure stress of each beam is also shown in the figures. It should be noted the measured stresses in the spliced bars, corresponding to the applied load level was obtained from the cracked-section analysis as discussed in section 4.1.

Figure 18: Propagation of the splitting cracks on the top surface of beam 11-8-O-C2
Figure 19: Flexural crack width of beam 8-5-O-C2 with No. 8 spliced bars

Figure 20: Flexural crack width of beam 11-8-X-C2 with No. 11 spliced bars
It is readily seen from Figures 19 and 20 that after cracking of the concrete section, the crack width had an increasing trend which is approximately linear up to stress level in the bars of 120 ksi. When the stress level reached 120 ksi the crack width increased dramatically with slight increase in the stress in the spliced bar. This behavior is anticipated due to the highly non-linear behavior of the steel beyond the stress level of 120 ksi and the significant reduction of its modulus of elasticity.
6. Predictions According to Codes

The ACI 318-05 code\(^4\) equation (ACI 318-05) and the equation proposed by the ACI committee 408\(^1\) (ACI 408R-03) were used to predict the stresses in the spliced bars.

Table 7: Codes predictions of stresses in confined and unconfined spliced bars

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Measured Stress</th>
<th>ACI 318-05 Stress</th>
<th>ACI 318-05 Ratio</th>
<th>ACI 408R-03 Stress</th>
<th>ACI 408R-03 Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8-5-O-C0</td>
<td>96</td>
<td>80</td>
<td>1.20</td>
<td>84</td>
<td>1.15</td>
</tr>
<tr>
<td></td>
<td>8-5-O-C1</td>
<td>140</td>
<td>80</td>
<td>1.75</td>
<td>104</td>
<td>1.34</td>
</tr>
<tr>
<td></td>
<td>8-5-O-C2</td>
<td>152</td>
<td>80</td>
<td>1.90</td>
<td>104</td>
<td>1.46</td>
</tr>
<tr>
<td>2</td>
<td>8-5-X-C0</td>
<td>110</td>
<td>104</td>
<td>1.06</td>
<td>103</td>
<td>1.07</td>
</tr>
<tr>
<td></td>
<td>8-5-X-C1</td>
<td>152</td>
<td>104</td>
<td>1.46</td>
<td>130</td>
<td>1.17</td>
</tr>
<tr>
<td></td>
<td>8-5-X-C2</td>
<td>152</td>
<td>104</td>
<td>1.46</td>
<td>130</td>
<td>1.17</td>
</tr>
<tr>
<td>3</td>
<td>8-8-O-C0</td>
<td>91</td>
<td>98</td>
<td>0.93</td>
<td>81</td>
<td>1.12</td>
</tr>
<tr>
<td></td>
<td>8-8-O-C1</td>
<td>151</td>
<td>122</td>
<td>1.24</td>
<td>124</td>
<td>1.21</td>
</tr>
<tr>
<td>4</td>
<td>8-8-X-C0</td>
<td>109</td>
<td>145</td>
<td>0.75</td>
<td>107</td>
<td>1.02</td>
</tr>
<tr>
<td></td>
<td>8-8-X-C1</td>
<td>152</td>
<td>182</td>
<td>0.84</td>
<td>155</td>
<td>0.98</td>
</tr>
<tr>
<td>5</td>
<td>11-5-O-C0</td>
<td>74</td>
<td>92</td>
<td>0.80</td>
<td>82</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td>11-5-O-C1</td>
<td>132</td>
<td>119</td>
<td>1.11</td>
<td>122</td>
<td>1.08</td>
</tr>
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<td>11-5-O-C2</td>
<td>151</td>
<td>119</td>
<td>1.27</td>
<td>148</td>
<td>1.02</td>
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<td>6</td>
<td>11-5-X-C0</td>
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<td>95</td>
<td>0.76</td>
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<td></td>
<td>11-5-X-C1</td>
<td>127</td>
<td>137</td>
<td>0.93</td>
<td>130</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>11-5-X-C2</td>
<td>155</td>
<td>137</td>
<td>1.13</td>
<td>165</td>
<td>0.94</td>
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<td>7</td>
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<td>79</td>
<td>0.98</td>
<td>75</td>
<td>1.04</td>
</tr>
<tr>
<td></td>
<td>11-8-O-C1</td>
<td>116</td>
<td>79</td>
<td>1.46</td>
<td>103</td>
<td>1.12</td>
</tr>
<tr>
<td></td>
<td>11-8-O-C2</td>
<td>152</td>
<td>79</td>
<td>1.92</td>
<td>103</td>
<td>1.48</td>
</tr>
<tr>
<td>8</td>
<td>11-8-X-C0</td>
<td>96</td>
<td>123</td>
<td>0.78</td>
<td>101</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td>11-8-X-C1</td>
<td>128</td>
<td>123</td>
<td>1.04</td>
<td>141</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td>11-8-X-C2</td>
<td>157</td>
<td>123</td>
<td>1.28</td>
<td>141</td>
<td>1.11</td>
</tr>
</tbody>
</table>
Table 7 provides the predicted stresses in the spliced bars for all test beams and the measured-prediction ratios according to the two predictive equations. Table 8 provides the maximum, minimum, and average of measured-prediction ratios, along with the standard deviation and coefficient of variation for the two equations.

Table 8: Statistics of codes prediction of splice strength

<table>
<thead>
<tr>
<th></th>
<th>ACI 318-05</th>
<th>ACI 408R-03</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average</td>
<td>1.18</td>
<td>1.09</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>0.36</td>
<td>0.17</td>
</tr>
<tr>
<td>Coefficient of Variation</td>
<td>0.30</td>
<td>0.16</td>
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<tr>
<td>Maximum</td>
<td>1.92</td>
<td>1.48</td>
</tr>
<tr>
<td>Minimum</td>
<td>0.69</td>
<td>0.76</td>
</tr>
</tbody>
</table>

It is clear from Tables 7 and 8 that ACI 408R-03 equation provides slightly better prediction of the stresses in the spliced bars in comparison to ACI 318-05 equation as indicated by the average of measured-prediction ratios. In addition, ACI 408R-03 equation exhibits less scatter than ACI 318-05 equation as demonstrated by the lower coefficient of variation. However, closer examination of the measured-prediction ratios according to the two expressions given in Table 7 suggests that both equations underestimate the effect of confinement by transverse reinforcement. Therefore, prediction of the stresses in unconfined spliced bars is presented separately from confined bars. Table 9 gives the predicted stresses and the measured-prediction ratios for unconfined spliced bars only according to both equations. On the other hand, the predicted stresses and the measured-prediction ratios for confined spliced bars are given in Table 10. Both tables also include the maximum, minimum, standard deviation, and coefficient of variation of the ratios. It should be noted that beams with confined spliced bars failing in splitting only were considered in Table 10 and the five beams failing in flexure were excluded.
Table 9: Codes predictions of stresses in unconfined spliced bars

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Measured Stress, ksi</th>
<th>ACI 318-05 Stress, ksi</th>
<th>ACI 318-05 Ratio</th>
<th>ACI 408R-03 Stress, ksi</th>
<th>ACI 408R-03 Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8-5-O-C0</td>
<td>96</td>
<td>80</td>
<td>1.20</td>
<td>84</td>
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</tr>
<tr>
<td>2</td>
<td>8-5-X-C0</td>
<td>110</td>
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<td>1.07</td>
</tr>
<tr>
<td>3</td>
<td>8-8-O-C0</td>
<td>91</td>
<td>98</td>
<td>0.93</td>
<td>81</td>
<td>1.12</td>
</tr>
<tr>
<td>4</td>
<td>8-8-X-C0</td>
<td>109</td>
<td>145</td>
<td>0.75</td>
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<td>1.02</td>
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<tr>
<td>5</td>
<td>11-5-O-C0</td>
<td>74</td>
<td>92</td>
<td>0.80</td>
<td>82</td>
<td>0.91</td>
</tr>
<tr>
<td>6</td>
<td>11-5-X-C0</td>
<td>72</td>
<td>105</td>
<td>0.69</td>
<td>95</td>
<td>0.76</td>
</tr>
<tr>
<td>7</td>
<td>11-8-O-C0</td>
<td>78</td>
<td>79</td>
<td>0.98</td>
<td>75</td>
<td>1.04</td>
</tr>
<tr>
<td>8</td>
<td>11-8-X-C0</td>
<td>96</td>
<td>123</td>
<td>0.78</td>
<td>101</td>
<td>0.96</td>
</tr>
</tbody>
</table>

Average: 0.90 1.00
Standard Deviation: 0.17 0.13
Coefficient of Variation: 0.19 0.13
Maximum: 1.20 1.15
Minimum: 0.69 0.76

It is readily seen from Table 9 that ACI 318-05 equation yields a slightly un-conservative estimate of unconfined spliced bars by producing an average measured-prediction ratio less than 1.0 and minimum ratio of 0.69. ACI 408R-03 equation yields a very reasonable conservative estimate of the stresses in the unconfined spliced bars. Furthermore, ACI 408R-03 exhibits less scatter than ACI 318-05 equation as demonstrated by the lower coefficient of variation which is also graphically represented in Figure 21. This behavior might be attributed to the how the concrete strength is accounted for in these two equations. ACI 318-05 equation represents the effect of concrete strength on the strength of unconfined splices by the square root, whereas ACI 408R-03 uses the fourth root. This renders the ACI 408R-03 expression as the more conservative expression to be used for estimating the bond strength of unconfined MMFX spliced bars, especially when higher concrete strength is used.
Figure 21: Distribution of measured-prediction ratios of unconfined spliced bars

Figure 22: Distribution of measured-prediction ratios of confined spliced bars
Table 10: Codes predictions of stresses in confined spliced bars

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Measured</th>
<th>Stresses in Confined Spliced Bars, ksi</th>
<th>ACI 318-05</th>
<th>ACI 408R-03</th>
</tr>
</thead>
<tbody>
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<td></td>
<td>Stress</td>
<td>Ratio</td>
<td>Stress</td>
<td>Ratio</td>
</tr>
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<td>8-5-O-C1</td>
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<td>80</td>
<td>1.75</td>
<td>104</td>
</tr>
<tr>
<td>3</td>
<td>8-8-O-C1</td>
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<td>1.24</td>
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</tr>
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<td>4</td>
<td>8-8-X-C1</td>
<td>152</td>
<td>182</td>
<td>0.84</td>
<td>155</td>
</tr>
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<td>5</td>
<td>11-5-O-C1</td>
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</tr>
<tr>
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<td>119</td>
<td>1.27</td>
<td>148</td>
</tr>
<tr>
<td>6</td>
<td>11-5-X-C1</td>
<td>127</td>
<td>137</td>
<td>0.93</td>
<td>130</td>
</tr>
<tr>
<td></td>
<td>11-5-X-C2</td>
<td>155</td>
<td>137</td>
<td>1.13</td>
<td>165</td>
</tr>
<tr>
<td>7</td>
<td>11-8-O-C1</td>
<td>116</td>
<td>79</td>
<td>1.46</td>
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</tr>
<tr>
<td>8</td>
<td>11-8-X-C1</td>
<td>128</td>
<td>123</td>
<td>1.04</td>
<td>141</td>
</tr>
</tbody>
</table>

Average: 1.19 1.06  
Standard Deviation: 0.28 0.14  
Coefficient of Variation: 0.23 0.13  
Maximum: 1.75 1.34  
Minimum: 0.83 0.91

It is evident from Table 10 both equations underestimate the effect of confining the spliced bars by transverse reinforcement. ACI 318-05 equation severely underestimates the effect of confinement, whereas ACI 408R-03 equation slightly underestimates the effect of confinement as indicated by the average of the measured-prediction ratios. In addition, ACI 408R-03 exhibits less scatter than ACI 318-05 as indicated by the much lower coefficient of variation and clearly demonstrated by Figure 22. However, both equations still can be safely used to estimate the bond strength of confined MMFX bars. This behavior is believed to be due to the fact that these expressions were derived based on test data with conventional steel with limit to yield strength of 80 ksi. It has been reported by other researchers that the stresses in the confining stirrups was as low as 9 ksi even for heavily confined spliced bars. When MMFX spliced bars are confined they utilize the transverse reinforcement more effectively by increasing the stresses in the stirrups due to the higher tension forces achieved in the bars.
7. Analysis of Test Results

7.1. Overview

The measured stresses were used to investigate the effect of the different parameters believed to affect the bond characteristics of the MMFX Steel. From the test results, it is shown that increasing the confinement level increases the load-carrying capacity and the ductility of the members. The measured stresses in the spliced bars not confined by transverse reinforcement were used to evaluate the effect of the development length and the concrete cover. To eliminate the effect of the concrete compressive strength, the measured stresses were normalized to the fourth root of the concrete compressive strength. The fourth root was chosen because it provides a better representation of the effect of concrete strength on bond behavior as reported by ACI committee 408 (ACI 408R-03).

7.2. Effect of the Splice Length

To study the effect of the splice length ($l_s$), the stresses developed in the unconfined spliced bars of the beams having the same cross-section were compared with each other as shown in Table 11. It is evident from Table 11 that increasing the splice length increases the developed stresses in the spliced bars. However, this increase is not proportional indicating that as the splice length increases, the effectiveness of the splice decreases. For No. 8 bars, increasing the splice length by 32 and 35 percent increased the splice strength by 16 and 14 percent, respectively. Similarly, for No. 11 bars, increasing the splice length by 33 percent increased the strength by 15 percent only. However, for the fifth and sixth groups of No. 11 bars increasing the splice length from 69 to 91 (49 and 65 times the bar diameter) did not significantly increase the stresses in the spliced bars. This behavior clearly indicates that as the splice length increases, they become less effective to the extent that very long splice lengths will not increase the bond capacity. This behavior was also observed by El-Hacha et al.\textsuperscript{5} who studied the bond strength of MMFX steel bars. Their test results revealed that providing a splice length of 80 times the bar diameter did not add to the bond strength. This behavior is
attributed to the well established fact the bond stress distribution is non-linear over long splice length as shown in Figure 23. While the assumption of constant bond stress distribution is reasonably accurate for short splice lengths, it becomes un-conservative for long splice lengths. Examining the increase in the stresses developed in the spliced bars to the length of the splice, it can be assumed that the splice strength is proportional to the square root of the ratio of splice length to bar diameter \( \sqrt{\frac{l_s}{d_b}} \).

Table 11: Effect of splice length on bond strength of MMFX steel bars

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Cover in.</th>
<th>( f'_{c} ) psi</th>
<th>( l_s ) in.</th>
<th>Increase in ( l_s ) %</th>
<th>Measured Stresses, ( f_s ) ksi</th>
<th>( \frac{f_s}{\sqrt{f'_{c}}} ) psi</th>
<th>Increase in strength %</th>
<th>Increase in ( \sqrt{\frac{l_s}{d_b}} ) %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8-5-O-C0</td>
<td>2.5</td>
<td>6015</td>
<td>31</td>
<td>32</td>
<td>96</td>
<td>10901</td>
<td>1.16</td>
<td>1.15</td>
</tr>
<tr>
<td>2</td>
<td>8-5-X-C0</td>
<td>2.5</td>
<td>5817</td>
<td>41</td>
<td>32</td>
<td>110</td>
<td>12596</td>
<td>1.16</td>
<td>1.15</td>
</tr>
<tr>
<td>3</td>
<td>8-8-O-C0</td>
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<td>8400</td>
<td>40</td>
<td>35</td>
<td>91</td>
<td>9505</td>
<td>1.14</td>
<td>1.16</td>
</tr>
<tr>
<td>4</td>
<td>8-8-X-C0</td>
<td>1.5</td>
<td>10200</td>
<td>54</td>
<td>35</td>
<td>109</td>
<td>10846</td>
<td>1.14</td>
<td>1.16</td>
</tr>
<tr>
<td>5</td>
<td>11-5-O-C0</td>
<td>2</td>
<td>5344</td>
<td>69</td>
<td>32</td>
<td>74</td>
<td>8655</td>
<td>1.04</td>
<td>1.15</td>
</tr>
<tr>
<td>6</td>
<td>11-5-X-C0</td>
<td>2</td>
<td>4058</td>
<td>91</td>
<td>32</td>
<td>72</td>
<td>9021</td>
<td>1.04</td>
<td>1.15</td>
</tr>
<tr>
<td>7</td>
<td>11-8-O-C0</td>
<td>3</td>
<td>6070</td>
<td>43</td>
<td>33</td>
<td>78</td>
<td>8837</td>
<td>1.14</td>
<td>1.15</td>
</tr>
<tr>
<td>8</td>
<td>11-8-X-C0</td>
<td>3</td>
<td>8383</td>
<td>57</td>
<td>33</td>
<td>96</td>
<td>10033</td>
<td>1.14</td>
<td>1.15</td>
</tr>
</tbody>
</table>

Figure 23: Distribution of bond stresses along splice length
7.3. **Effect of the Concrete Cover**

To investigate the effect of the concrete cover, the measured stresses were normalized to the square root of the ratio of the splice length and the bar diameter to eliminate the effect of the splice length as given in Table 12. It is readily seen from Table 12 that increasing the concrete cover increases the splice strength. However, this relation is not linear as revealed by the direct comparison between the increase in the cover and the increase in the normalized stresses. Increasing the concrete cover by 67 percent increased the normalized splice strength by 30 percent. Similarly, increasing the cover by 50 percent increased the normalized stresses by 29 percent only. This behavior is expected as the distribution of tensile stresses on the concrete cover is not constant. Further investigation led to the conclusion that it is fairly accurate to represent the effect of the concrete cover on the normalized splice strength by the square root of the ratio of the thickness of the concrete cover to the diameter of the reinforcing bar.

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Cover in.</th>
<th>Increase in Cover %</th>
<th>$f'_c$ psi</th>
<th>$l_s$ in.</th>
<th>Measured Stresses, $f_s$ ksi</th>
<th>Increase in strength %</th>
<th>Increase in $\sqrt{\frac{c}{d_b}}$ %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8-5-O-C0</td>
<td>2.5</td>
<td>67</td>
<td>6015</td>
<td>31</td>
<td>96</td>
<td>1958</td>
<td>1.30</td>
</tr>
<tr>
<td></td>
<td>8-8-O-C0</td>
<td>1.5</td>
<td></td>
<td>8400</td>
<td>40</td>
<td>91</td>
<td>1503</td>
<td>1.29</td>
</tr>
<tr>
<td>2</td>
<td>8-5-X-C0</td>
<td>2.5</td>
<td>67</td>
<td>5817</td>
<td>41</td>
<td>110</td>
<td>1967</td>
<td>1.33</td>
</tr>
<tr>
<td></td>
<td>8-8-X-C0</td>
<td>1.5</td>
<td></td>
<td>10200</td>
<td>54</td>
<td>109</td>
<td>1476</td>
<td>1.29</td>
</tr>
<tr>
<td>5</td>
<td>11-5-O-C0</td>
<td>2</td>
<td>50</td>
<td>5344</td>
<td>69</td>
<td>74</td>
<td>1237</td>
<td>1.29</td>
</tr>
<tr>
<td>7</td>
<td>11-8-O-C0</td>
<td>3</td>
<td></td>
<td>6070</td>
<td>43</td>
<td>78</td>
<td>1600</td>
<td>1.22</td>
</tr>
<tr>
<td>6</td>
<td>11-5-X-C0</td>
<td>2</td>
<td>50</td>
<td>4058</td>
<td>91</td>
<td>72</td>
<td>1123</td>
<td>1.41</td>
</tr>
<tr>
<td>8</td>
<td>11-8-X-C0</td>
<td>3</td>
<td></td>
<td>8383</td>
<td>57</td>
<td>96</td>
<td>1578</td>
<td>1.22</td>
</tr>
</tbody>
</table>
7.4. Proposed Splice Strength Equation for Unconfined Bars

After evaluating the effect of the various parameters on the developed stresses, the measured stresses in the unconfined spliced bars were normalized with respect to the concrete strength, development length, and concrete cover. The relationships derived above were used to isolate the effect of these parameters. Therefore, the measured stresses in the unconfined spliced bars of each beam of the four groups was divided by the fourth root of the concrete strength \( \sqrt[4]{f'_c} \), square root of ratio of the splice length to the bar diameter \( \sqrt{\frac{l_s}{d_b}} \), and the square root of the ratio of the concrete cover to the bar diameter \( \sqrt{\frac{c}{d_b}} \) as shown in Table 13. In addition, to the test results reported herein, Table 13 contains the test results provided by the University of Kansas and the University of Texas at Austin on their research. These results were used to further investigate this behavior and are graphically presented in Figure 24 to demonstrate the scatter of the normalized stresses.

![Figure 24: Scatter of the normalized test results](image-url)
Table 13: Normalized stresses in unconfined spliced bars from the test results

<table>
<thead>
<tr>
<th>University</th>
<th>Beam ID</th>
<th>$d_b$ in.</th>
<th>Cover in.</th>
<th>$f_c'$ psi</th>
<th>Splice Length in.</th>
<th>Measured Stresses ksi</th>
<th>$\frac{f_c'}{\sqrt{d_b} \sqrt{c}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>NCSU</td>
<td>8-5-O-C0-2.5</td>
<td>1</td>
<td>2.5</td>
<td>6015</td>
<td>31</td>
<td>96</td>
<td>1238</td>
</tr>
<tr>
<td></td>
<td>8-5-X-C0-2.5</td>
<td>1</td>
<td>2.5</td>
<td>5817</td>
<td>41</td>
<td>110</td>
<td>1244</td>
</tr>
<tr>
<td></td>
<td>8-8-O-C0-1.5</td>
<td>1</td>
<td>1.5</td>
<td>8400</td>
<td>40</td>
<td>91</td>
<td>1227</td>
</tr>
<tr>
<td></td>
<td>8-8-X-C0-1.5</td>
<td>1</td>
<td>1.5</td>
<td>10200</td>
<td>54</td>
<td>109</td>
<td>1205</td>
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<tr>
<td></td>
<td>11-5-O-C0-2.0</td>
<td>1.41</td>
<td>2</td>
<td>5344</td>
<td>69</td>
<td>74</td>
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<tr>
<td></td>
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<td>1.41</td>
<td>2</td>
<td>4058</td>
<td>91</td>
<td>72</td>
<td>943</td>
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<tr>
<td></td>
<td>11-8-O-C0-3.0</td>
<td>1.41</td>
<td>3</td>
<td>6070</td>
<td>43</td>
<td>78</td>
<td>1097</td>
</tr>
<tr>
<td></td>
<td>11-8-X-C0-3.0</td>
<td>1.41</td>
<td>3</td>
<td>8383</td>
<td>57</td>
<td>96</td>
<td>1082</td>
</tr>
<tr>
<td>UT</td>
<td>5-5-O-C0-3/4</td>
<td>0.625</td>
<td>0.75</td>
<td>5200</td>
<td>33</td>
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<td>1184</td>
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<tr>
<td></td>
<td>5-5-X-C0-3/4</td>
<td>0.625</td>
<td>0.75</td>
<td>5200</td>
<td>44</td>
<td>91</td>
<td>1166</td>
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<tr>
<td></td>
<td>5-5-O-C0-1.25</td>
<td>0.625</td>
<td>1.25</td>
<td>5200</td>
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<td>88</td>
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<td></td>
<td>5-5-X-C0-1.25</td>
<td>0.625</td>
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<td>5200</td>
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<td>110</td>
<td>1448</td>
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<td></td>
<td>5-5-O-C0-2.0</td>
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<td>5700</td>
<td>15</td>
<td>97</td>
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<td>5700</td>
<td>25</td>
<td>120</td>
<td>1221</td>
</tr>
<tr>
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<td>8-8-OC0-1.5</td>
<td>1</td>
<td>1.5</td>
<td>8300</td>
<td>40</td>
<td>80</td>
<td>1082</td>
</tr>
<tr>
<td></td>
<td>8-5-OC0-1.5</td>
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<td>1.5</td>
<td>5200</td>
<td>40</td>
<td>72</td>
<td>1095</td>
</tr>
<tr>
<td></td>
<td>8-8-XC0-1.5</td>
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<td>1.5</td>
<td>7800</td>
<td>54</td>
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<td>8-5-OC0-1.5</td>
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<td>1.5</td>
<td>5000</td>
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<td>74</td>
<td>1048</td>
</tr>
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<td></td>
<td>8-5-XC0-1.5</td>
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<td>1.5</td>
<td>4700</td>
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<td>82</td>
<td>1027</td>
</tr>
<tr>
<td></td>
<td>11-5-OC0-3.0</td>
<td>1.41</td>
<td>2.75</td>
<td>5000</td>
<td>50</td>
<td>75</td>
<td>1072</td>
</tr>
<tr>
<td></td>
<td>11-5-XC0-3.0</td>
<td>1.41</td>
<td>2.75</td>
<td>5400</td>
<td>67</td>
<td>84</td>
<td>1018</td>
</tr>
<tr>
<td>KU</td>
<td>8-5-O-C0-1.5</td>
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<td>1.4</td>
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<td>47</td>
<td>77</td>
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<td></td>
<td>8-5-X-C0-1.5</td>
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<td>1.29</td>
<td>4880</td>
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<td>1048</td>
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<td>1.41</td>
<td>5940</td>
<td>63</td>
<td>88</td>
<td>1064</td>
</tr>
</tbody>
</table>
It is seen from Table 13 that the value of the normalized stresses for the 24 test specimens with spliced bars not confined by transverse reinforcement are approximately equal. This indicates that the proposed relationships between the splice strength and the concrete strength, splice length, concrete cover, and bar diameter can very accurately represent the effect of these parameters on the bond strength of MMFX steel bars.

The average of the 24 tested specimens is 1100 with a coefficient of variation of 0.11. Based on the limited data presented herein the stresses in the spliced bars not confined by transverse reinforcement can be calculated using units of psi and in. as follows:

$$f_s = \frac{1100 \sqrt{f_c'} \sqrt{l_s} \sqrt{c}}{d_b} \text{ (psi)}$$

Where;

- $f_s$ = stresses in the spliced bars, psi
- $f_c'$ = concrete compressive strength, psi
- $l_s$ = splice length, in.
- $c$ = minimum clear cover, in.
- $d_b$ = diameter of spliced bars, in.
8. Conclusions

Based on the research findings, the following conclusions can be drawn:

1. Splitting of concrete cover was the prevailing mode of failure for the spliced test beams except for five beams failed due to flexure. Use of excess amount of transverse reinforcement to confine the spliced bars resulted in an increase in bond force causing the beams to fail in flexural mode.

2. The failure of beams with spliced bars not confined by transverse reinforcement was very violent and explosive associated with scattering and spalling of the concrete cover over the entire splice length.

3. The use of transverse reinforcement to confine the spliced bars allows the failure to be more gradual with visible splitting cracks in the concrete cover prior to failure.

4. Based on the test results, a stress level of 90 and 70 ksi can be achieved by No. 8 and No. 11 MMFX spliced bars without the use of transverse reinforcement.

5. Confining the spliced bars by transverse reinforcement increased the stresses developed reaching a stress level of 150 ksi for No. 8 and No. 11 MMFX bars. In addition, the use of transverse reinforcement to confine the spliced bars increased the ultimate load and ductility of the beams.

6. The presence of stirrups to confine the spliced bars limited the progress of the splitting cracks; however, they did not delay the initiation of the splitting cracks. Splitting cracks were initiated when the stress in the No. 8 and No. 11 spliced bars reached a value of approximately 70 and 40 ksi, respectively, roughly in inverse ratio of the bars areas.
7. Flexural cracks were formed first at both ends of the splice, then within the splice zone. After cracking the width of flexural cracks increased proportionally to the stress level in the splice bars up to 120 ksi. When the stresses in the spliced bars exceed 120 ksi the width of flexural crack increased significantly with slight increase in the stress level.

8. Increasing the splice length increased the strength of the splice; however the increase was not proportional. It was demonstrated that it is fairly accurate to assume that the splice strength is proportional to the square root of the ratio of the splice length and the bar diameter ($\sqrt{L_s/d_b}$).

9. Increasing the concrete cover increases the stress developed in the spliced bars. However, the relationship between the cover and the splice strength is not linear and it was concluded that this relation can be best represented by the square root of the ratio of the cover to the bar diameter ($\sqrt{c/d_b}$).

10. ACI 318-05 equation uses the square root of the concrete strength to account for effect of concrete strength on bond strength. Therefore, it provides an un-conservative prediction of the stresses in unconfined spliced bars, especially when higher concrete strength is considered.

11. ACI committee 408 equation uses the fourth root of the concrete strength to represent its influence on strength of unconfined spliced bars. Therefore, the equation yields more conservative prediction and less scatter in comparison to the ACI 318-05 code equation.
12. ACI 318-05 equation underestimates the effect of confinement on bond strength, more than ACI committee 408. However, both equations can be safely used to estimate the bond strength of spliced MMFX bars confined by transverse reinforcement.

13. The proposed equation in Section 7.4 can be used to predict the stresses in MMFX spliced bars not confined by transverse reinforcement.
9. References

1. ACI Committee 408, 2003, “Bond and Development of Straight Reinforcing in Tension,” (ACI 408R-03), American Concrete Institute, Farmington Hills, Michigan, 49 p.


3. Concrete Innovations Appraisal Service (CIAS), 2003, “Structural Design Criteria for High-Strength MMFX Microcomposite Reinforcing Bars,” A Service of the Strategic Development Council (SDC) of the Concrete Research and Educational Foundation (ConREF), Report 04-1, 34 p.

4. ACI Committee 318, 2005, “Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05),” American Concrete Institute, Farmington Hills, Michigan, 430 p.

Appendix
Appendix: Detailed Test Results

A.1 General

The load-deflection behavior and mode of failure of each group of beams are given in the following sections. In addition the measured flexural crack width versus measured stresses in the spliced bars is also given. It should be noted that he plotted deflection is the total deflection at mid-span of the beam with respect to the deflected position of the end of the beam. Also, the measured stresses in the spliced bars are obtained from cracked-section analysis of the tested beams using the measured applied load as discussed in section 4.1.

A.2 Test Results of the First Group

The first group of beams (beams 8-5-O-C0, 8-5-O-C1, and 8-5-O-C2) had a development length of 31 in., concrete cover of 2.5 in., and measured concrete strength at the day of testing of 6015 psi. The load-deflection behavior is shown in Figure A.1. The three beams at the conclusion of the test are shown in Figure A.2 through Figure A.4. In addition, the measured flexural crack width versus measured stresses in the spliced bars is shown in Figure A.5.
Figure A.1: Load-deflection behavior of beams of the first group (8-5-O-C0, C1, and C2)

Figure A.2: Beam 8-5-O-C0 at the conclusion of the test
Figure A.3: Beam 8-5-O-C1 at the conclusion of the test

Figure A.4: Beam 8-5-O-C2 at the conclusion of the test
A.3 Test Results of the Second Group

The second group of beams (beams 8-5-X-C0, 8-5-X-C1, and 8-5-X-C2) had the same concrete cover as the first group of 2.5 in., but development length of 41 in. was used and the measured concrete strength at the day of testing was 5817 psi. The load-deflection behavior is shown in Figure A.6. The three beams at failure are shown in Figure A.7 through Figure A.9. Moreover, the measured flexural crack width versus measured stresses in the spliced bars is shown in Figure A.10.

Figure A.5: Flexural crack width of the beam 8-5-O-C2 of the first group
Figure A.6: Load-deflection behavior of beams of second groups (8-5-X-C0, C1, and C2)

Figure A.7: Beam 8-5-X-C0 at the conclusion of the test
Figure A.8: Beam 8-5-X-C1 at the conclusion of the test

Figure A.9: Beam 8-5-X-C2 at the conclusion of the test
A.4 Test Results of the Third Group

The third group of beams (beams 8-8-O-C0 and 8-8-O-C1) had a development length of 40 in., concrete cover of 1.5 in., and measured concrete strength at the day of testing of 8400 psi. The load-deflection behavior of the beams is shown in Figure A.11. The two beams at the conclusion of the test are shown in Figure A.12 and Figure A.13. Moreover, the measured flexural crack width versus measured stresses in the spliced bars is shown in Figure A.14.
Figure A.11: Load-deflection behavior of beams of third group (8-8-O-C0 and C1)

Figure A.12: Beam 8-8-O-C0 at the conclusion of the test
Figure A.13: Beam 8-8-O-C1 at the conclusion of the test

Figure A.14: Flexural crack width of the beam 8-8-O-C1 of the third group
A.5 Test Results of the Fourth Group

The fourth group of beams (beams 8-8-X-C0 and 8-8-X-C1) had the same concrete cover as the third group of 1.5 in., but development length of 54 in. and measured concrete strength at the day of testing of 10200 psi. The load-deflection behavior of the beams is shown in Figure A.15. The two beams at the conclusion of the test are shown in Figure A.16 and Figure A.17. In addition, the measured flexural crack width versus measured stresses in the spliced bars is shown in Figure A.18.

![Figure A.15: Load-deflection behavior of beams of fourth group (8-8-X-C0 and C1)](image-url)
Figure A.16: Beam 8-8-X-C0 at the conclusion of the test

Figure A.17: Beam 8-8-X-C1 at the conclusion of the test
A.6 Test Results of the Fifth Group

The fifth group of beams (beams 11-5-O-C0, 11-5-O-C1 and 11-5-O-C2) had a development length of 69 in., concrete cover of 2.0 in., and measured concrete strength at the day of testing of 5344 psi. The load-deflection behavior of the beams is shown in Figure A.19. The three beams at the conclusion of the test are shown in Figure A.20 through Figure A.22. Moreover, the measured flexural crack width versus measured stresses in the spliced bars is shown in Figure A.23.
Figure A.19: Load-deflection behavior of beams of the fifth group (11-5-O-C0, C1, and C2)

Figure A.20: Beam 11-5-O-C0 at the conclusion of the test
Figure A.21: Beam 11-5-O-C1 at the conclusion of the test

Figure A.22: Beam 11-5-O-C2 at the conclusion of the test
A.7 Test Results of the Sixth Group

The sixth group of beams (beams 11-5-X-C0, 11-5-X-C1 and 11-5-X-C2) had the same concrete cover as the fifth group of 2.0 in., but development length of 91 in. and measured concrete strength at the day of testing of 4058 psi. The load-deflection behavior of the beams is shown in Figure A.24. The three beams at the conclusion of the test are shown in Figure A.25 through Figure A.27. In addition, the measured flexural crack width versus measured stresses in the spliced bars is shown in Figure A.28.
Figure A.24: Load-deflection behavior of beams of the sixth group (11-5-X-C0, C1, and C2)

Figure A.25: Beam 11-5-X-C0 at the conclusion of the test
Figure A.26: Beam 11-5-X-C1 at the conclusion of the test

Figure A.27: Beam 11-5-X-C2 at the conclusion of the test
A.8 Test Results of the Seventh Group

The seventh group of beams (beams 11-8-O-C0, 11-8-O-C1, and 11-8-O-C2) had a development length of 43 in., concrete cover of 3.0 in., and measured concrete strength at the day of testing of 6070 psi. The load-deflection behavior is shown in Figure A.29. The three beams at the conclusion of the test are shown in Figure A.30 through Figure A.32. In addition, the measured flexural crack width versus measured stresses in the spliced bars is shown in Figure A.33.
Figure A.29: Load-deflection behavior of beams of the seventh group (11-8-O-C0, C1, and C2)

Splice Length = 43 in.

Figure A.30: Beam 11-8-O-C0 at the conclusion of the test
Figure A.31: Beam 11-8-O-C1 at the conclusion of the test

Figure A.32: Beam 11-8-O-C2 at the conclusion of the test
A.9 Test Results of the Eighth Group

The eighth group of beams (beams 11-8-X-C0, 11-8-X-C1, and 11-8-X-C2) had the same concrete cover as the seventh group of 3.0 in., but development length of 57 in. was used and the measured concrete strength at the day of testing was 8383 psi. The load-deflection behavior is shown in Figure A.34. The three beams at failure are shown in Figure A.35 through Figure A.37. Moreover, the measured flexural crack width versus measured stresses in the spliced bars is shown in Figure A.38.
Figure A.34: Load-deflection behavior of beams of the eighth group (11-8-X-C0, C1, and C2)

Figure A.35: Beam 11-8-X-C0 at the conclusion of the test
Figure A.36: Beam 11-8-X-C1 at the conclusion of the test

Figure A.37: Beam 11-8-X-C2 at the conclusion of the test
Figure A.38: Flexural crack width of the beam 11-8-X-C2 of the eighth group