BOND BEHAVIOR OF MMFX (ASTM A 1035) REINFORCING STEEL

A Summary Report

Of a Cooperative Research Program – Phase I

Submitted to

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FOREWORD

In structural concrete design, adequate bond between the reinforcing steel and concrete is essential. The current ACI code provisions for bond and development length of reinforcement are empirical relationships based on the reports of ACI Committee 408 and other publications in the literature. Although ACI 408 has an extensive database, virtually all the data were obtained from tests using reinforcement with specified yield strength no more than 80 ksi. It is uncertain whether the current code provisions are applicable for reinforcement with much higher yield strength.

MMFX steel reinforcement is a newer product, which is characterized by its high tensile strength and linear behavior up to stress level of 100 ksi without a well-defined yield plateau. To use this reinforcement efficiently for concrete structures, it is necessary to conduct research to determine whether the current code provisions are applicable for MMFX reinforcement and, if not, to develop new design recommendations.

A cooperative research program on bond behavior of MMFX reinforcing steel was organized by North Carolina State University (NCSU), in partnership with the University of Kansas (KU), and the University of Texas at Austin (UT). Being able to conduct independent tests concurrently at three institutions made it possible to develop research data more rapidly with greater reliability and confidence. In total, sixty-six tests were conducted using large tension-spliced beam specimens

This summary report provides a brief description of the research program and presents the research findings and recommendations. Detailed discussions of the research are documented in several publications prepared by different authors at the three institutions. These publications are listed in the appendix and can be obtained without charge from the indicated Web sites.

ACKNOWLEDGMENT

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The contents of this report reflect the views of the authors and not necessarily the views of the sponsor. The authors are solely responsible for the accuracy of the data obtained, the observation and interpretation of the experimental evidence, and the findings and recommendations.

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EXECUTIVE SUMMARY

Findings and Recommendations

For the sixty-six specimens tested, splitting of the concrete cover was the prevailing mode of failure except for five specimens tested by NCSU, which failed in flexure. Failure of specimens with unconfined spliced bars was sudden in an abrupt manner. Use of transverse reinforcement to confine the spliced bars produced more gradual failure accompanied by visible concrete splitting cracks prior to failure.

Test results indicated that, with appropriate splice length, the top and side covers, and bar spacings as used in the test specimens of this study, a maximum stress level of 120, 110 and 96 ksi could be developed in No. 5, No. 8 and No. 11 MMFX spliced bars, respectively, without the use of transverse reinforcement (see Tables 5, 6, and 7). By confining the MMFX spliced bars with transverse reinforcement, the stresses developed by No. 8 and No. 11 bars were increased to an average of 150 ksi (see Tables 6 and 7). Use of transverse reinforcement also increased the ultimate load and the deformation capacities of the tested specimens. Therefore, whenever possible it is recommended that MMFX spliced bars be confined by transverse reinforcement to fully utilize their strength and to improve the deformation capacity of the member with splices.

Based on a statistical evaluation of the test data and the average of the ratios between the developed and calculated values to assess the current bond equations, it was determined that ACI 318-05 code design equation overestimates the strength of unconfined spliced MMFX bars, especially for high strength concrete. On the other hand, the bond equation for design recommended by ACI Committee 408 (as best-fit to the database but including a strength-reduction factor ϕ of 0.82) underestimates the stresses for unconfined spliced bars for all but two out of 31 cases, but with less scatter than those obtained using the ACI 318-05 equation. The statistical evaluation of the test data and the average of the ratios between the developed and calculated values using both the ACI 318-05 and ACI Committee 408 equations suggest that both equations can be used to compute the bond strength of spliced MMFX bars confined by transverse reinforcement. Again, the ACI Committee 408 equation is more conservative than the ACI 318-05 equation. Accordingly, the ACI Committee 408 equation with a strength-reduction factor ϕ of 0.82 is recommended for development and splice design using MMFX steel.

Scope of Research

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

Bar size:	No. 5, No. 8, and No. 11
Target Concrete Compressive Strength:	5000 and 8000 psi
Concrete Cover:	¾ in., 1¼ in., and 2.0 in. for No. 5 bars 1.5 in. and 2.5 in. for No. 8 bars 2.0 in. 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 confining transverse reinforcement
Confinement Level:	First level (C1) to provide 20 ksi increase over unconfined splice length Second level (C2) to provide 40 ksi increase over unconfined splice length Third level (C3) to provide 80 ksi increase over unconfined splice length

The entire 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 specimens to provide crosschecks amongst the three universities. These common specimens are highlighted in Table 1.

Table 1: Collective test matrix for the three universities

<table>
<tr><th>f_c’
ksi</th><th>Bar
Size</th><th colspan="6">University of Kansas
(KU)</th><th colspan="6">North Carolina State
University (NCSU)</th><th colspan="6">University of Texas at
Austin (UT)</th></tr>
<tr><td rowspan="9">5</td><td rowspan="3">5</td><td colspan="6">Cover (in.)</td><td colspan="6">Cover (in.)</td><td colspan="6">Cover (in.)</td></tr>
<tr><td colspan="2">¾</td><td colspan="2">1¼</td><td colspan="2">2.0</td><td colspan="2">¾</td><td colspan="2">1¼</td><td colspan="2">2.0</td><td colspan="2">¾</td><td colspan="2">1¼</td><td colspan="2">2.0</td></tr>
<tr><td colspan="2">O-C0
X-C0</td><td colspan="2">O-C0
X-C0</td><td colspan="2"></td><td colspan="2"></td><td colspan="2"></td><td colspan="2"></td><td colspan="2">O-C0
X-C0</td><td colspan="2">O-C0
X-C0</td><td colspan="2">O-C0
X-C0</td></tr>
<tr><td rowspan="3">8</td><td colspan="6">Cover (in.)</td><td colspan="6">Cover (in.)</td><td colspan="6">Cover (in.)</td></tr>
<tr><td colspan="3">1.5</td><td colspan="3">2.5</td><td colspan="3">1.5</td><td colspan="3">2.5</td><td colspan="3">1.5</td><td colspan="3">2.5</td></tr>
<tr><td colspan="3">O-C0,1,2
X-C0,1,2</td><td colspan="3"></td><td colspan="3"></td><td colspan="3">O-C0,2,3
X-C0,2,3</td><td colspan="3">O-C0,2
X-C0,2</td><td colspan="3"></td></tr>
<tr><td rowspan="3">11</td><td colspan="6">Cover (in.)</td><td colspan="6">Cover (in.)</td><td colspan="6">Cover (in.)</td></tr>
<tr><td colspan="3">2.0</td><td colspan="3">3.0</td><td colspan="3">2.0</td><td colspan="3">3.0</td><td colspan="3">2.0</td><td colspan="3">3.0</td></tr>
<tr><td colspan="3"></td><td colspan="3"></td><td colspan="3">O-C0,2,3
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X-C0,1,2</td></tr>
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X-C0,1,2</td><td colspan="3">O-C0,2
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<tr><td colspan="3">O-C0,1,2
X-C0,1,2</td><td colspan="3"></td><td colspan="3"></td><td colspan="3">O-C0,2,3
X-C0,2,3</td><td colspan="3"></td><td colspan="3"></td></tr>
<tr><td colspan="2">Total</td><td colspan="6">22</td><td colspan="6">22</td><td colspan="6">22</td></tr>
</table>

* In addition, Hoyt and Donnelly at UT tested additional specimens that were outside the scope of this research program (see Appendix). However, the results of three of these additional UT tests are included in this report.

The design of the splice lengths to achieve the required stresses in the bars was calculated according to the bond equation recommended by ACI Committee 408 (Equation 4-11a, ACI 408R-03), but using a strength-reduction factor (ϕ-factor) of 1.0. Similarly, the amount of transverse reinforcement required to achieve the desired stresses in the spliced bars was determined according to the same equation. ACI Committee 408 bond equation is as follows:

$$\frac{l_d}{d_b} = \frac{\left(\frac{f_s}{\phi f_c'^{1/4}} - 2400\omega\right)\alpha\beta\lambda}{76.3\left(\frac{c\omega + K_{tr}}{d_b}\right)} \qquad \text{Equation (1)}$$

Where

l_d	=	development or splice length (in.)
d_b	=	diameter of bar (in.)
f_s	=	stress in reinforcing bar (psi)
f_c'	=	compressive strength of concrete (psi)
ω	=	$0.1\ c_{max}/c_{min} + 0.9 \leq 1.25$
c	=	$c_{min} + 0.5d_b$ (in.)
c_{max}	=	maximum of c_b and c_s (in.)
c_{min}	=	minimum of c_b and c_s (in.)
c_b	=	clear bottom cover for bar being developed or spliced (in.)
c_s	=	minimum of c_{so} and c_{si}+0.25 in. (in.)
c_{so}	=	clear side cover for bar being developed or spliced (in.)
c_{si}	=	one-half of the bars clear spacing (in.)
K_{tr}	=	transverse reinforcement index
	=	$\left(\frac{0.52 t_r t_d A_{tr}}{sn}\right)\sqrt{f_c'}$

$$\left(\frac{c\omega + K_{tr}}{d_b}\right) \leq 4$$

t_r = term representing the effect of relative rib area on bond strength

= $9.6R_r + 0.28 \leq 1.72$

R_r = relative rib area of the bar (0.0727 for conventional reinforcement)

t_d = term representing the effect of bar size on bond strength

= $0.78d_b + 0.22$

A_{tr} = total cross-sectional area of all transverse reinforcement within spacing "s" that crosses the potential plane of splitting through the reinforcement being developed or spliced (in.^{2}).

s = center-to-center spacing of transverse reinforcement (in.).

n = number of bars being developed or spliced.

α = reinforcement location factor

= 1.3 for reinforcement placed so that more than 12 in. (300 mm) of fresh concrete is cast below the development length or splice and 1.0 for other reinforcement.

β = coating factor

= 1.0 for uncoated bars, 1.5 for epoxy-coated bars with cover less than $3d_b$, or clear spacing less than $6d_b$, and 1.2 for other epoxy-coated bars.

$\alpha\beta \leq 1.7$

λ = lightweight concrete factor

= 1.3 for lightweight concrete and 1.0 for normalweight concrete.

A five-part notation system was developed to identify the tested specimens. The notation of the specimens used in Table 1 and hereafter is shown in Figure 1.

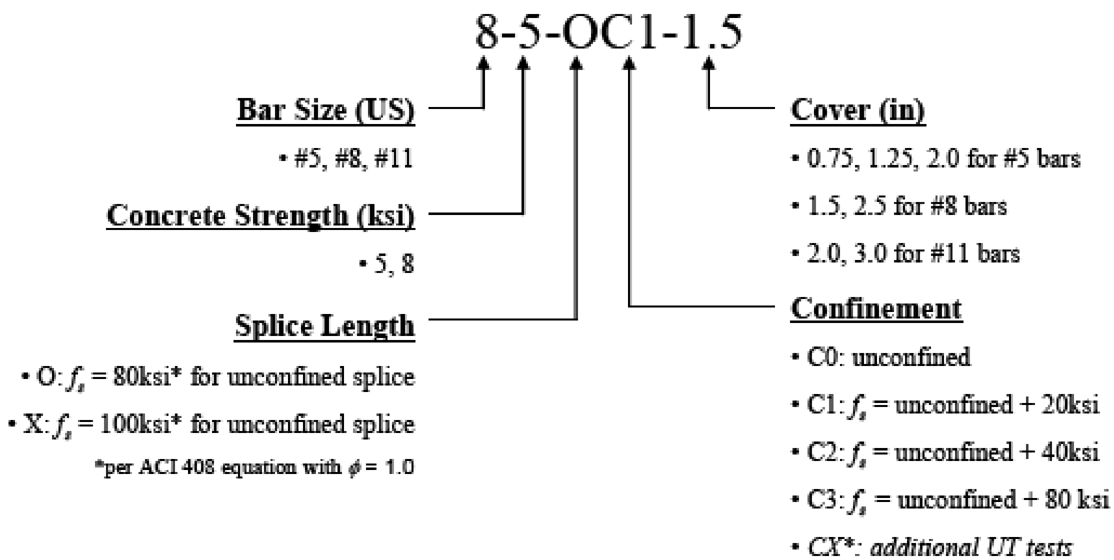

Figure 1: Notation system

Research Methodology

Test Specimens

Large-scale beam-splice specimens were used to study the bond characteristics of MMFX steel reinforcing bars to concrete. Beam-splice specimens are recommended by ACI Committee 408 since they provide the most realistic state of stress in comparison to other test configurations. In beam-splice specimens 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 bottom concrete covers, as well as clear bars spacing equal to twice the selected concrete cover as shown in Figure 2.

The details of the specimens with No.5, No. 8, and No. 11 MMFX bars are given in Tables 2, 3, and 4, respectively. Beam specimens (with No. 8 and No. 11 bars) contained two splices only, while slab specimens (with No. 5 bars) contained four splices as shown in Figure 2. Duplicate beams are highlighted in the tables using the same color. For the duplicate specimens, the target stress represents a nominal value to be used in designing the test specimen. Slight differences in details such as size of cross-section, tie spacing, and splice length are possible.

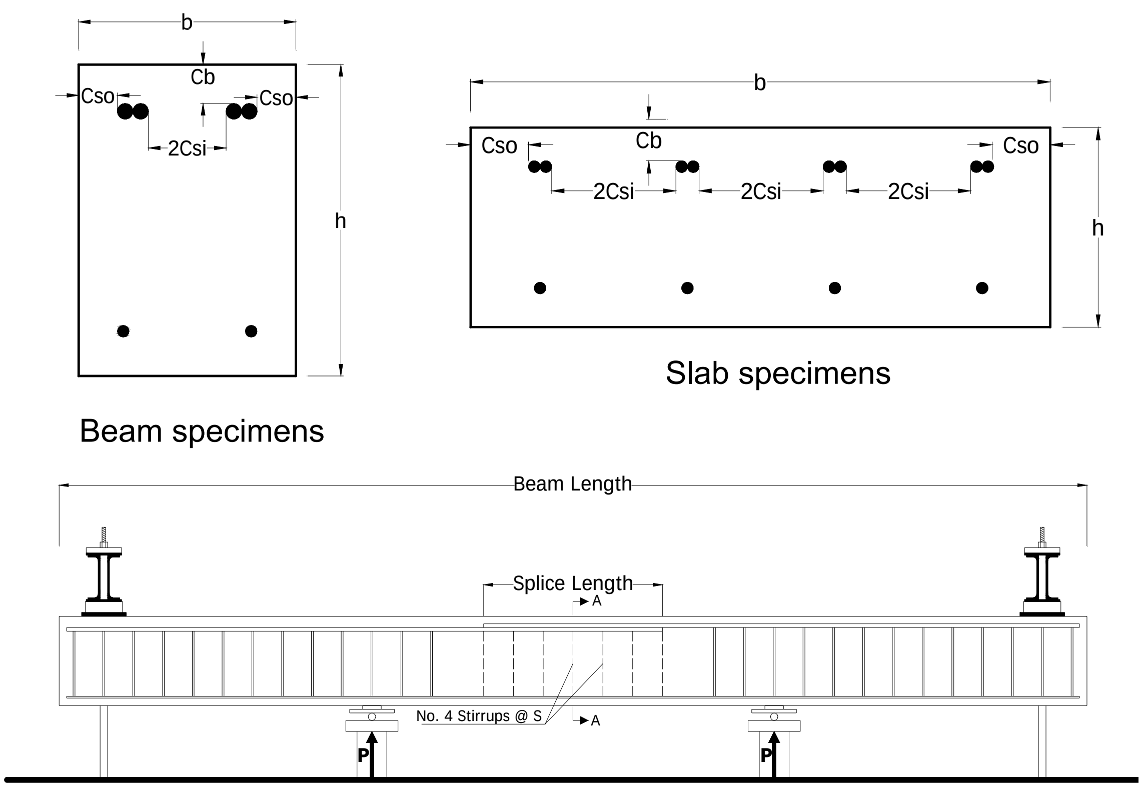

Figure 2: Details of beam-splice specimens

Table 2: Details of beam-splice specimens with No. 5 MMFX bars

Specimen ID	f'_c	Beam Length	Cross Section	Cover	Splice Length	Stirrup Spacing	Target Stress
	psi	ft.	in.	in.	in.	in.	ksi
University of Texas at Austin							
5-5-O-C0-3/4	5000	14	13 x 12	0.75	32	N/A	80
5-5-X-C0-3/4					43		100
5-5-O-C0-1¼			35 x 12	1.25	18		80
5-5-X-C0-1¼					25		100
5-5-O-C0-2.0			35 x 12	2.00	15		80
5-5-X-C0-2.0					20		100
University of Kansas							
5-5-O-C0-3/4	5000	15	14 x 20	0.75	32	N/A	80
5-5-X-C0-3/4					43		100
5-5-O-C0-1¼			35 x 10	1.25	18		80
5-5-X-C0-1¼					25		100

Table 3: Details of beam-splice specimens with No. 8 MMFX bars

Specimen ID	f'c	Beam Length	Cross Section	Cover	Splice Length	Stirrup Spacing	Target Stress
	psi	ft.	in.	in.	in.	in.	ksi
University of Texas at Austin							
8-5-O-C0-1.5	5000	18	10 x 27	1.50	47	N/A	80
8-5-O-C2-1.5						5.5	120
8-5-X-C0-1.5					62	N/A	100
8-5-X-C2-1.5						7.0	140
8-5-O-C0*-1.5					40	N/A	80
8-5-O-C1*-1.5						13.5	100
8-5-O-C2*-1.5						7.0	120
8-8-O-C0-1.5	8000		10 x 23	1.50	40	N/A	80
8-8-O-C1-1.5						13.5	100
8-8-O-C2-1.5						7.0	120
8-8-X-C0-1.5			10 x 27		54	N/A	100
8-8-X-C1-1.5						18.5	120
8-8-X-C2-1.5						9.0	140
University of Kansas							
8-5-O-C0-1.5	5000	21	14 x 30	1.50	47	N/A	80
8-5-O-C1-1.5						11.75	100
8-5-O-C2-1.5						5.88	120
8-5-X-C0-1.5					63	N/A	100
8-5-X-C1-1.5						15.75	120
8-5-X-C2-1.5						7.88	140
8-8-O-C0-2.5	8000		14 x 21	2.50	27	N/A	80
8-8-O-C1-2.5						13.50	100
8-8-O-C2-2.5						5.38	120
8-8-X-C0-2.5					36	N/A	100
8-8-X-C1-2.5						18.00	120
8-8-X-C2-2.5						7.25	140
North Carolina State University							
8-5-O-C0-2.5	5000	23	14 x 24	2.50	31	N/A	80
8-5-O-C2-2.5						4.00	120
8-5-O-C3-2.5						2.00	160
8-5-X-C0-2.5					41	N/A	100
8-5-X-C2-2.5						5.00	140
8-5-X-C3-2.5						2.50	>160
8-8-O-C0-1.5	8000		10 x 24	1.50	40	N/A	80
8-8-O-C2-1.5						7.50	120
8-8-X-C0-1.5					54	N/A	100
8-8-X-C2-1.5						10.50	140

Table 4: Details of beam-splice specimens with No. 11 MMFX bars

Specimen ID	f'_c	Beam Length	Cross Section	Cover	Splice Length	Stirrup Spacing	Target Stress
	psi	ft.	in.	in.	in.	in.	ksi
University of Texas at Austin							
11-5-O-C0-3.0	5000	22	18 x 31	3.00	50	N/A	80
11-5-O-C1-3.0						8.00	100
11-5-O-C2-3.0						4.00	120
11-5-X-C0-3.0					67	N/A	100
11-5-X-C1-3.0						11.00	120
11-5-X-C2-3.0						5.50	140
University of Kansas							
11-8-O-C0-2.0	8000	24	24 x 26	2.00	58	N/A	80
11-8-O-C1-2.0						14.50	100
11-8-O-C2-2.0						6.50	120
11-8-X-C0-2.0					79	N/A	100
11-8-X-C1-2.0						19.75	120
11-8-X-C2-2.0						8.75	140
North Carolina State University							
11-5-O-C0-2.0	5000	23	14 x 36	2.00	69	N/A	80
11-5-O-C2-2.0						6.50	120
11-5-O-C3-2.0						3.00	160
11-5-X-C0-2.0					91	N/A	100
11-5-X-C2-2.0						8.00	140
11-5-X-C3-2.0						4.00	>160
11-8-O-C0-3.0	8000		18x24	3.00	43	N/A	80
11-8-O-C2-3.0						5.50	120
11-8-O-C3-3.0						2.50	160
11-8-X-C0-3.0					57	N/A	100
11-8-X-C2-3.0						7.00	140
11-8-X-C3-3.0						3.50	>160

It should be noted that the specimens were cast with the spliced bars in the bottom of the form to avoid the top-bar effect. Prior to testing, the specimens were rotated 180 degrees about their longitudinal axes to place the spliced bars at the top to facilitate mapping the cracks and test observations.

Test Setup

All specimens were tested in four-point bending to develop a constant moment zone where the spliced bars were located. At NCSU and UT the load was applied using hydraulic jacks reacting against the strong floor and the test specimens were supported by tying down stiff steel beams to the floor using prestressing bars. At KU, the specimens were supported at the interior points and loads were applied at the ends of the specimens by pulling downward through the strong floor. A picture of the typical test setup used for testing is shown in Figure 3.

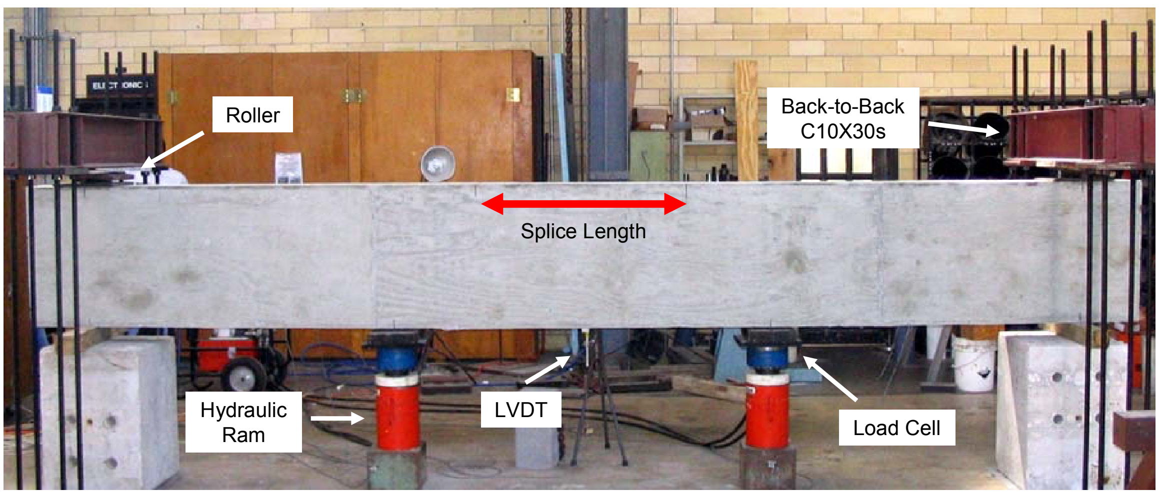

Figure 3: Typical test setup (UT)

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. Displacement transducers were used to measure the deflection at mid-span and at supports. 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 test data.

Test Results

General

The stress-strain relationships measured at UT produced the following three exponential equations for modeling MMFX steel bars. These equations were used in all subsequent computations:

$$f_s = 156\left(1 - e^{-220\varepsilon_s}\right) \qquad \text{for No. 5 bars}$$

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

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

Stresses Developed in Spliced Bars

The stresses developed in the spliced bars were determined from cracked-section analysis of the tested specimens using the measured applied load and the aforementioned exponential equation for MMFX steel. The computed stresses, the measured concrete compressive strength on the day of testing, and the measured concrete covers are given in Tables 5, 6, and 7 for No. 5, No. 8, and No. 11 bars, respectively. Duplicate specimens for crosschecks are highlighted in the tables using the same color.

It is readily seen from Table 5 that increasing the concrete cover increased the stresses developed in the No. 5 MMFX bars while using shorter splice lengths. In addition, Tables 6 and 7 show that confining the No. 8 and No, 11 spliced bars using transverse reinforcement increased the stresses developed in the bars. It is evident that use 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 an increase in the stresses developed in the spliced bars.

Table 5: Stresses developed in No. 5 MMFX spliced bars

Specimen ID	f'_c	Concrete Cover			Splice Length	Stirrup Spacing	Developed Stress
		c_b	c_{so}	c_{si}			
	psi	in.	in.	in.	in.	in.	ksi
University of Texas at Austin							
5-5-O-C0-3/4	5200	0.75	1.00	1.00	33	N/A	80
5-5-X-C0-3/4	5200	0.75	1.00	1.00	44		91
5-5-O-C0-1¼	5200	1.25	3.50	3.75	18		88
5-5-X-C0-1¼	5200	1.25	3.50	3.75	25		110
5-5-O-C0-2.0	5700	2.00	3.50	3.75	15		97
5-5-X-C0-2.0	5700	2.00	3.50	3.75	20		120
University of Kansas							
5-5-O-C0-3/4	5490	0.80	1.11	1.15	32	N/A	77
5-5-X-C0-3/4	4670	0.70	0.96	1.21	43		82
5-5-O-C0-1¼	5490	1.09	3.72	3.76	18		87
5-5-X-C0-1¼	4670	0.98	3.80	3.73	25		91

The bar stresses achieved during the tests indicate that a maximum stress level of 120, 110, and 96 ksi could be developed by No. 5, No. 8 and No. 11 MMFX spliced bars, respectively, without the use of transverse reinforcement. These maximum stress levels achieved were dependent on the concrete strength, concrete cover, and splice length used. By confining the MMFX spliced bars with transverse reinforcement, the stresses developed by No. 8 and No. 11 bars were increased to an average of 150 ksi.

For No. 11 MMFX bars, a splice length of 65 bar diameter (NCSU: 11-5-X-C0-2.0) did not enhance the stresses developed in the bars, indicating that using long splice lengths without confinement is an inefficient way to achieve high stress levels. Therefore, it is recommended that shorter splice lengths with confinement provided by transverse reinforcement should be used rather than longer splice lengths without confining steel. In addition, the use of couplers to splice high strength steel bars should be investigated as a more economic alternative, especially when high stress levels are to be developed without the use of transverse reinforcement.

Table 6: Stresses developed in No. 8 MMFX spliced bars

Specimen ID	f'c	Concrete Cover			Splice Length	Stirrups Spacing	Developed Stress
		c_b	c_{so}	c_{si}			
	psi	in.	in.	in.	in.	in.	ksi
University of Texas at Austin							
8-5-O-C0-1.5	5000	1.50	1.55	1.45	47	N/A	74
8-5-O-C2-1.5	5000	1.50	1.65	1.38		5.22	141
8-5-X-C0-1.5	4700	1.50	1.50	1.50	62	N/A	82
8-5-X-C2-1.5	4700	1.50	1.60	1.38		6.89	148
8-5-O-C0*-1.5	5200	1.50	1.55	1.45	40	N/A	72
8-5-O-C1*-1.5	5200	1.50	1.65	1.38		13.33	99
8-5-O-C2*-1.5	5200	1.50	1.65	1.38		6.67	129
8-8-O-C0-1.5	8300	1.50	1.60	1.40	40	N/A	80
8-8-O-C1-1.5	8300	1.50	1.65	1.38		13.33	123
8-8-O-C2-1.5	8300	1.50	1.65	1.38		6.67	147
8-8-X-C0-1.5	7800	1.50	1.50	1.50	54	N/A	86
8-8-X-C1-1.5	7800	1.50	1.50	1.50		18.00	122
8-8-X-C2-1.5	7800	1.50	1.50	1.50		9.00	144
University of Kansas							
8-5-O-C0-1.5	5260	1.40	1.48	3.60	47	N/A	78
8-5-O-C1-1.5	4720	1.60	1.57	3.47		11.75	124
8-5-O-C2-1.5	6050	1.40	1.50	3.58		5.88	127
8-5-X-C0-1.5	5940	1.41	1.41	3.69	63	N/A	90
8-5-X-C1-1.5	4720	1.50	1.58	3.42		15.75	129
8-5-X-C2-1.5	5010	1.50	1.55	3.45		7.88	143
8-8-O-C0-2.5	8660	2.30	2.31	2.79	27	N/A	80
8-8-O-C1-2.5	7790	2.44	2.26	2.97		13.50	89
8-8-O-C2-2.5	7990	2.17	2.31	2.77		5.38	115
8-8-X-C0-2.5	7990	2.38	2.44	2.67	36	N/A	91
8-8-X-C1-2.5	7790	2.56	2.39	2.71		18.00	111
8-8-X-C2-2.5	8660	2.31	2.48	2.57		7.25	117
North Carolina State University							
8-5-O-C0-2.5	6020	2.50	2.50	2.50	31	N/A	96
8-5-O-C2-2.5	6020	2.50	2.50	2.50		4.00	140
8-5-O-C3-2.5‡	6020	2.50	2.50	2.50		2.00	152
8-5-X-C0-2.5	5820	2.50	2.50	2.50	41	N/A	110
8-5-X-C2-2.5‡	5820	2.50	2.50	2.50		5.00	152
8-5-X-C3-2.5‡	5820	2.50	2.50	2.50		2.50	152
8-8-O-C0-1.5	8400	1.50	1.50	1.50	40	N/A	91
8-8-O-C2-1.5	8400	1.50	1.50	1.50		7.50	151
8-8-X-C0-1.5	10200	1.50	1.50	1.50	54	N/A	109
8-8-X-C2-1.5	10200	1.50	1.50	1.50		10.50	152

Table 7: Stresses developed in No. 11 MMFX spliced bars

Specimen ID	f'_c	Concrete Cover			Splice Length	Stirrup Spacing	Developed Stress
		c_b	c_{so}	c_{si}			
	psi	in.	in.	in.	in.	in.	ksi
University of Texas at Austin							
11-5-O-C0-3.0	5000	2.75	3.25	2.88	50	N/A	75
11-5-O-C1-3.0	5000	2.75	3.25	3.00		8.33	104
11-5-O-C2-3.0	5000	2.75	3.25	3.00		4.17	128
11-5-X-C0-3.0	5400	2.75	3.13	3.00	67	N/A	84
11-5-X-C1-3.0	5400	2.75	3.13	2.94		11.17	117
11-5-X-C2-3.0	5400	2.75	3.13	2.94		5.58	141
University of Kansas							
11-8-O-C0-2.0	9370	1.89	1.89	7.41	58	N/A	68
11-8-O-C1-2.0	9370	1.63	1.76	7.52		14.50	96
11-8-O-C2-2.0	8680	2.00	2.00	7.18		6.50	124
11-8-X-C0-2.0	9910	1.85	1.95	7.32	79	N/A	79
11-8-X-C1-2.0	9910	2.01	2.11	7.18		19.75	107
11-8-X-C2-2.0	8680	2.00	2.00	7.18		8.75	137
North Carolina State University							
11-5-O-C0-2.0	5340	2.00	2.00	2.00	69	N/A	74
11-5-O-C2-2.0	5340	2.00	2.00	2.00		6.50	132
11-5-O-C3-2.0	5340	2.00	2.00	2.00		3.00	151
11-5-X-C0-2.0	4060	2.00	2.00	2.00	91	N/A	72
11-5-X-C2-2.0	4060	2.00	2.00	2.00		8.00	127
11-5-X-C3-2.0	4060	2.00	2.00	2.00		4.00	155
11-8-O-C0-3.0	6070	3.00	3.00	3.00	43	N/A	78
11-8-O-C2-3.0	6070	3.00	3.00	3.00		5.50	116
11-8-O-C3-3.0‡	6070	3.00	3.00	3.00		2.50	152
11-8-X-C0-3.0	8380	3.00	3.00	3.00	57	N/A	96
11-8-X-C2-3.0	8380	3.00	3.00	3.00		7.00	128
11-8-X-C3-3.0‡	8380	3.00	3.00	3.00		3.50	157

‡ Beams failed in flexure by crushing of concrete in compression zone.

<u>Mode of Failure</u>

In general, failure due to splitting of the concrete cover was the prevailing mode of failure. However, five specimens tested by NCSU 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 an excessive amount of transverse reinforcement to confine the spliced bars in these five specimens resulted in an increase in bond force, and thus enabling flexural failure to occur.

Specimens with spliced bars not confined by transverse reinforcement failed very suddenly in an explosive and abrupt manner as shown in Figure 4. The specimens failed very shortly after the initiation of the splitting cracks with sudden loss of the load-carrying capacity. It was observed that the higher the failure load, the greater the likelihood that the splices would fail explosively. For slab specimens containing four splices, the exterior splices failed before the interior splices.

Use of transverse reinforcement to confine the spliced bars caused more gradual failure accompanied with fully visible splitting cracks in the concrete cover, thus giving advance warning. The confining stirrups limited the progress of the splitting cracks and enabled the specimen 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 at the top over the entire splice length.

Figure 4: Typical failure of specimens with unconfined spliced bars (NCSU)

Load-Deflection Behavior

The load-deflection behavior of the specimens reflects the effect of the splice strength on the ultimate load and deformation capacity of the specimen. A typical load-deflection behavior of test specimens containing No. 8 and No. 11 spliced bars is shown in Figures 5 and 6, respectively. The plotted deflection is the total deflection at mid-span of the specimen with respect to the ends of the specimen.

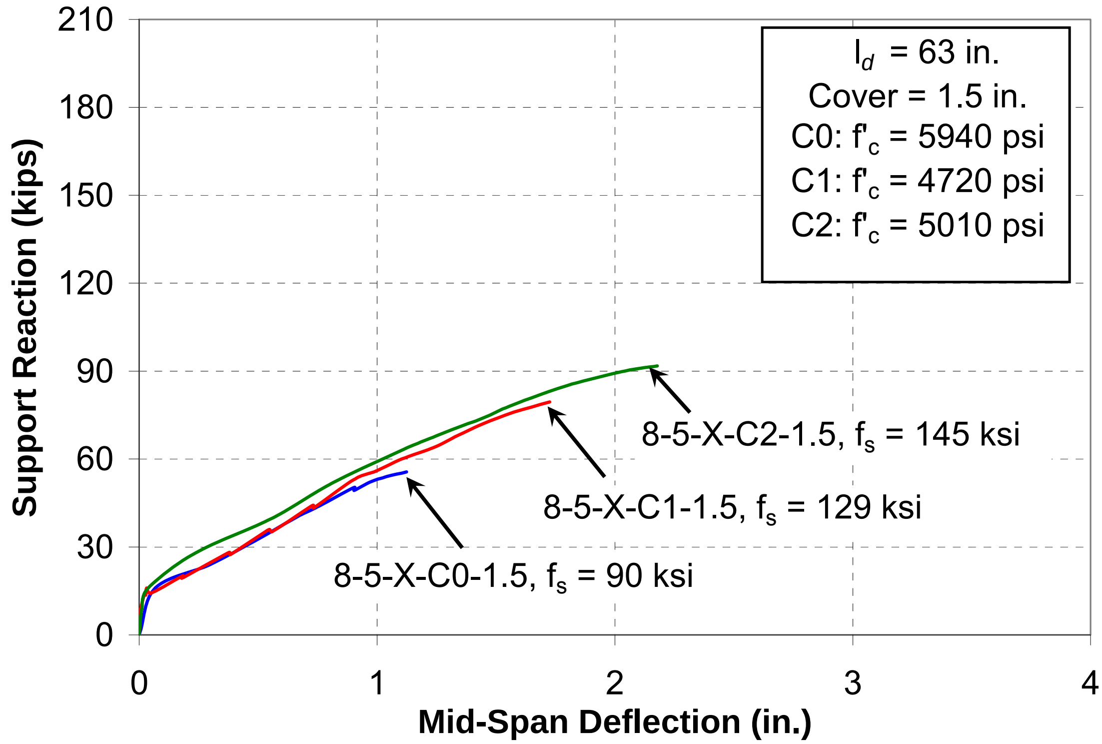

Figure 5: Load-deflection behavior of specimens with No. 8 bars (KU: 8-5-X-C0, C1, C2-1.5)

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 specimens. Specimens 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 specimens containing spliced bars not confined by transverse reinforcement failed in a very brittle manner at much lower load and significantly less deflection than the specimens with confined spliced bars. In addition, specimens with the first level of confinement exhibited less deflection and slightly less load at failure in comparison to specimens with second level of confinement.

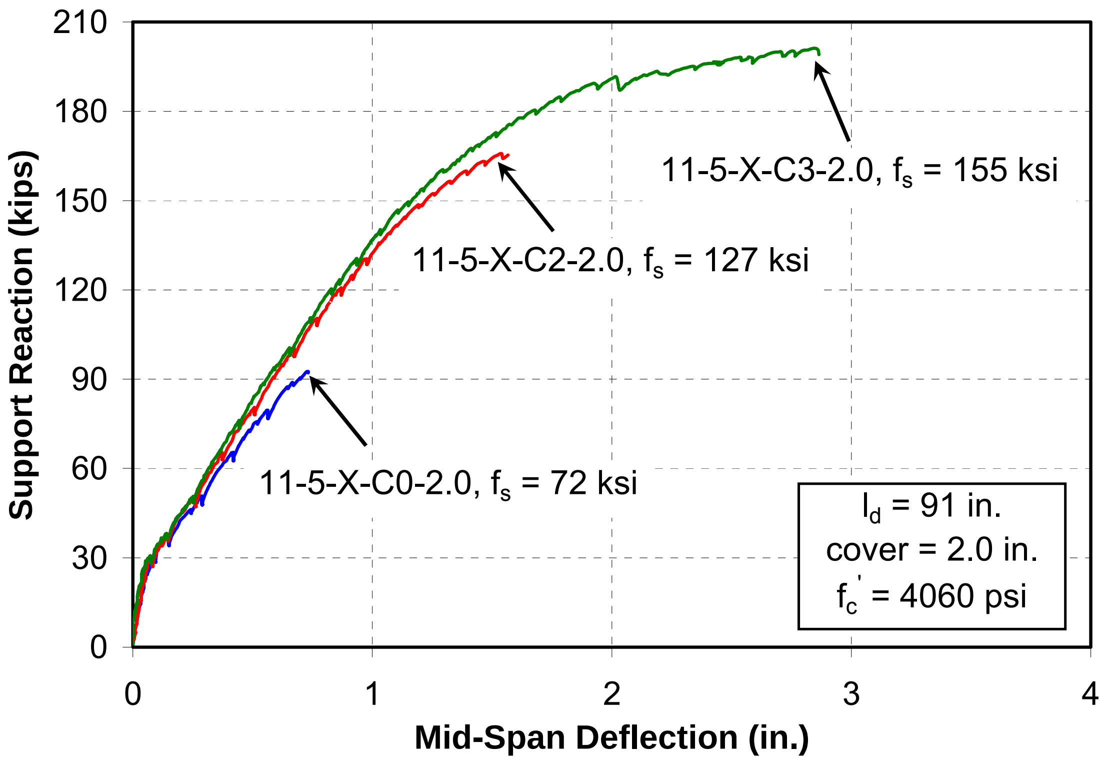

Figure 6: Load-deflection behavior of specimens with No. 11 bars (NCSU: 11-5-X-C0, C2, C3-2.0)

Crack Pattern

For all test specimens, 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 before they were observed inside the splice zone. Flexural cracks propagated downwards 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 specimen followed by splitting cracks on the side of the specimen at the level of the splices, terminating at the ends of the splice. However, the formation of the splitting cracks did not inhibit the flexural cracks from spreading and propagating towards the compression zone throughout loading until failure occurred. The presence of transverse reinforcement to confine the spliced bars prevented early failure and allowed the splitting cracks to become fully visible prior to failure, giving sufficient warning as shown in Figure 7.

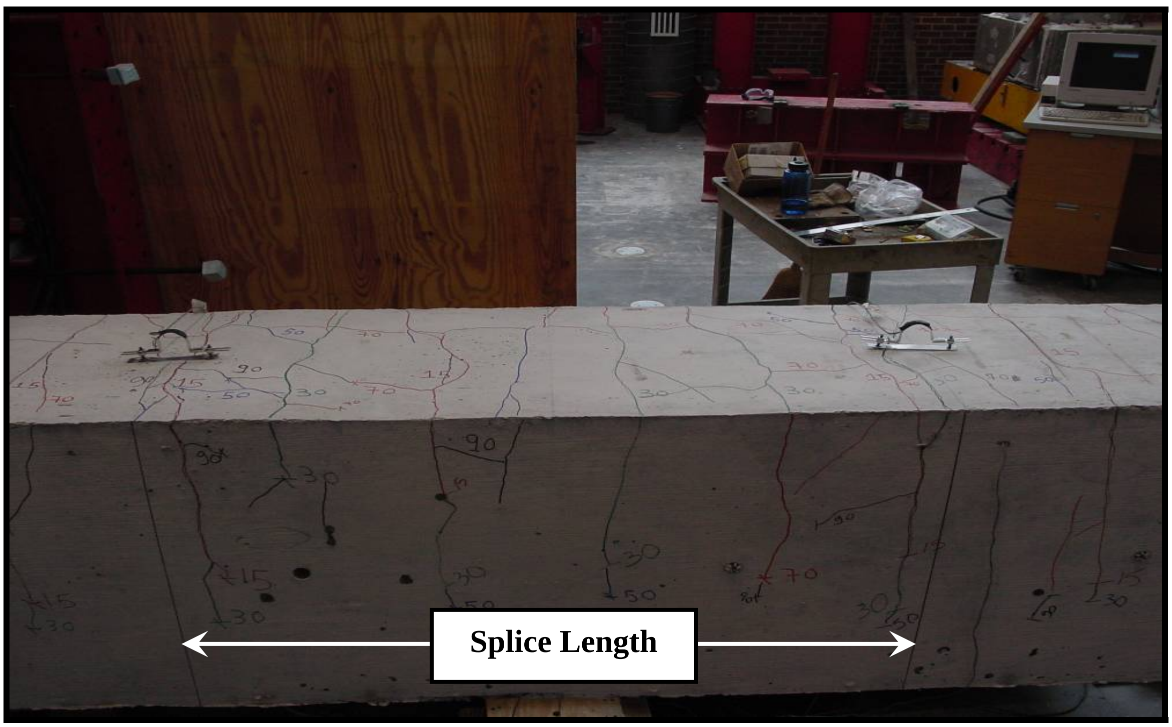

Figure 7: Propagation of the splitting cracks on the top surface of specimen11-8-O-C2-3.0 (NCSU)

Calculated Stresses

The ACI 318-05 code equation and the design equation recommended by the ACI Committee 408 (ACI 408R-03, Eq. 4-11a) were used to calculate the stresses in the spliced bars. The values calculated using the two equations for unconfined splices and confined splices are given in Tables 8 and 9, respectively. A strength-reduction factor (ϕ-factor) is not used in the ACI 318-05 equation since it is already included in the expression; while a ϕ-factor of 0.82 was used in the ACI Committee 408 equation. It should be noted that the five specimens that failed in flexure were excluded from Table 9. ACI 318-05 bond equation is as follows:

$$\frac{l_d}{d_b} = \frac{3}{40}\frac{f_y}{\sqrt{f_c'}}\frac{\psi_t \psi_e \psi_s \lambda}{\left(\frac{c_b + K_{tr}}{d_b}\right)} \qquad \text{Equation (2)}$$

Where

l_d = development or splice length (in.)

d_b = diameter of bar (in.)

f_s = stress in reinforcing bar (psi)

f_c' = compressive strength of concrete (psi)

Ψ_t = reinforcement location factor

= 1.3 for reinforcement placed so that more than 12 in. (300 mm) of fresh concrete is cast below the development length or splice and 1.0 for other reinforcement.

Ψ_e = coating factor

= 1.0 for uncoated bars, 1.5 for epoxy-coated bars with cover less than $3d_b$, or clear spacing less than $6d_b$, and 1.2 for all other epoxy-coated bars.

$\Psi_t\Psi_e \leq 1.7$

Ψ_s = bar size factor

= 0.8 for No. 6 and smaller bars and 1.0 for No. 7 bars and larger.

λ = lightweight concrete factor

= 1.3 for lightweight concrete and 1.0 for normalweight concrete..

c_b = smallest of the side cover and the cover over the bar (in both cases measured to the center of the bar), or one-half the center-to-center bar spacing of the bars (in.).

K_{tr} = transverse reinforcement index

$$= \frac{A_{tr} f_{yt}}{1500sn}$$

$$\left(\frac{c_b + K_{tr}}{d_b}\right) \leq 2.5$$

A_{tr} = total cross-sectional area of all transverse reinforcement within spacing “s” that crosses the potential plane of splitting through the reinforcement being developed (in.^{2}).

s = center-to-center spacing of transverse reinforcement (in.).

n = number of bars being developed or spliced.

The values in Table 8 show that the ACI 408R-03 equation underestimates the splice strength (average of developed/calculated values = 1.19), while the ACI 318-05 equation overestimates the splice strength (average of developed/calculated values = 0.87). In addition, the values calculated using the ACI 408R-03 equation exhibit less scatter (COV = 0.11) than those calculated using the ACI 318-05 equation (COV = 0.20) as demonstrated in Figure 8.

As shown in Table 9, both the ACI 408R-03 and ACI 318-05 equations underestimate the effect of confining the spliced bars by transverse reinforcement, with the former being slightly better than the latter (average of developed/calculated values = 1.29 versus 1.10), as indicated by the developed versus calculated ratios. In addition, the values using the ACI 408R-03 equation exhibit less scatter (COV = 0.10) than those by the ACI 318-05 equation (COV = 0.21), as demonstrated in Figure 9.

Since the ACI Committee 408 design equation is conservative for both unconfined and confined spliced bars, it is recommended that the ACI Committee 408 design equation with a ϕ-factor of 0.82 be used for development and splice design using MMFX steel with design parameters comparable to those used in this test program.

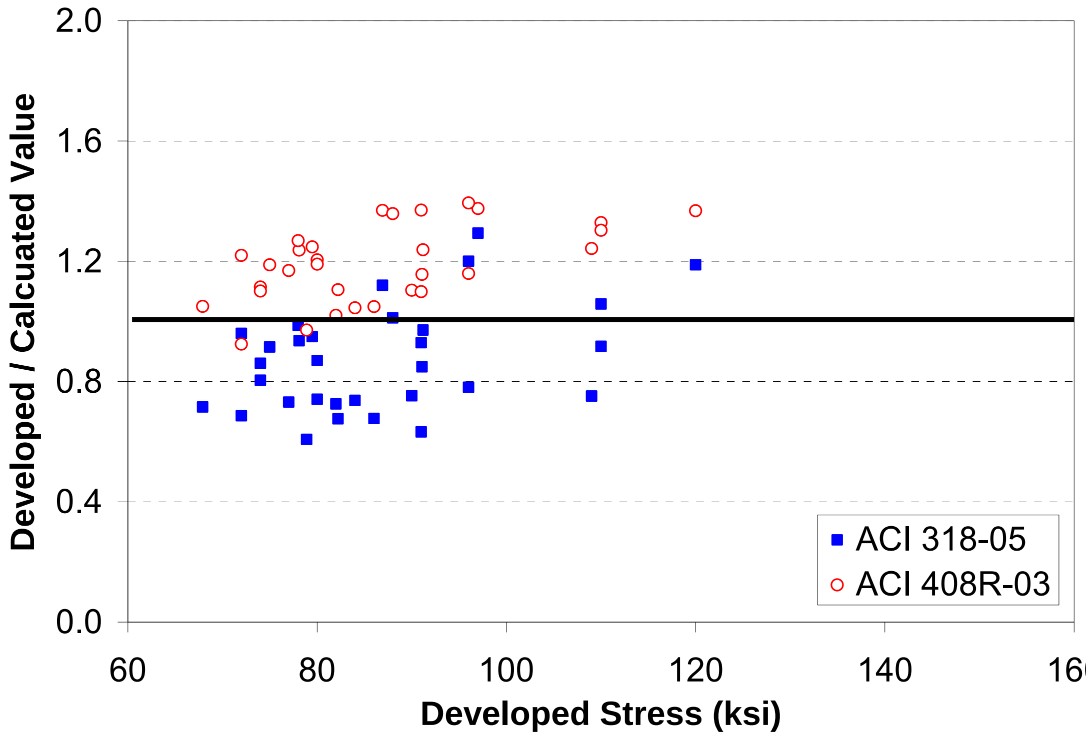

Figure 8: Distribution of developed/calculated values of unconfined splices

Table 8: Calculated stresses in unconfined splices

Specimen ID	Developed Stress ksi	ACI 318-05		ACI 408R-03	
		Stress ksi	Developed / Calculated	Stress ksi	Developed / Calculated
University of Kansas					
5-5-O-C0-3/4	77	105	0.73	66	1.17
5-5-X-C0-3/4	82	122	0.68	74	1.11
5-5-O-C0-1¼	87	78	1.12	63	1.37
5-5-X-C0-1¼	91	94	0.97	74	1.24
8-5-O-C0-1.5	78	84	0.94	63	1.24
8-5-X-C0-1.5	90	120	0.75	82	1.10
8-8-O-C0-2.5	80	84	0.95	64	1.25
8-8-X-C0-2.5	91	107	0.85	79	1.16
11-8-O-C0-2.0	68	95	0.72	65	1.05
11-8-X-C0-2.0	79	130	0.61	81	0.97
University of Texas at Austin					
5-5-O-C0-3/4	80	108	0.74	66	1.20
5-5-X-C0-3/4	91	144	0.63	83	1.10
5-5-O-C0-1¼	88	87	1.01	65	1.36
5-5-X-C0-1¼	110	120	0.92	83	1.33
5-5-O-C0-2.0	97	75	1.29	71	1.38
5-5-X-C0-2.0	120	101	1.19	88	1.37
8-5-O-C0-1.5	74	86	0.86	66	1.11
8-5-X-C0-1.5	82	113	0.73	80	1.02
8-5-O-C0*-1.5	72	75	0.96	59	1.22
8-8-O-C0-1.5	80	92	0.87	67	1.19
8-8-X-C0-1.5	86	127	0.68	82	1.05
11-5-O-C0-3.0	75	82	0.91	63	1.19
11-5-X-C0-3.0	84	114	0.74	80	1.05
North Carolina State University					
8-5-O-C0-2.5	96	80	1.20	69	1.39
8-5-X-C0-2.5	110	104	1.06	84	1.30
8-8-O-C0-1.5	91	98	0.93	66	1.37
8-8-X-C0-1.5	109	145	0.75	88	1.24
11-5-O-C0-2.0	74	92	0.80	67	1.10
11-5-X-C0-2.0	72	105	0.69	78	0.92
11-8-O-C0-3.0	78	79	0.99	62	1.27
11-8-X-C0-3.0	96	123	0.78	83	1.16
			0.87	AVG.	1.19
			0.18	ST. DEV.	0.13
			0.20	COV	0.11
			1.29	MAX	1.39
			0.61	MIN	0.92

Table 9: Calculated stresses in confined splices

Specimen ID	Developed Stress (ksi)	ACI 318-05		ACI 408R-03	
		Stress	Dev. / Calc.	Stress	Dev. / Calc.
University of Kansas					
8-5-O-C1-1.5	124	108	1.15	82	1.51
8-5-O-C2-1.5	127	122	1.04	104	1.22
8-5-X-C1-1.5	129	142	0.91	97	1.33
8-5-X-C2-1.5	143	149	0.96	111	1.29
8-8-O-C1-2.5	89	79	1.12	73	1.21
8-8-O-C2-2.5	115	80	1.43	83	1.39
8-8-X-C1-2.5	111	106	1.05	91	1.22
8-8-X-C2-2.5	117	112	1.05	106	1.11
11-8-O-C1-2.0	96	106	0.90	78	1.23
11-8-O-C2-2.0	124	128	0.97	100	1.23
11-8-X-C1-2.0	107	161	0.66	103	1.03
11-8-X-C2-2.0	137	164	0.84	115	1.19
University of Texas at Austin					
8-5-O-C2-1.5	141	111	1.27	103	1.36
8-5-X-C2-1.5	148	142	1.04	116	1.27
8-5-O-C1*-1.5	99	95	1.04	72	1.37
8-5-O-C2*-1.5	129	96	1.34	85	1.51
8-8-O-C1-1.5	123	120	1.03	85	1.44
8-8-O-C2-1.5	147	121	1.21	103	1.42
8-8-X-C1-1.5	122	155	0.79	99	1.23
8-8-X-C2-1.5	144	159	0.91	116	1.24
11-5-O-C1-3.0	104	84	1.24	80	1.31
11-5-O-C2-3.0	128	84	1.52	92	1.39
11-5-X-C1-3.0	117	116	1.01	97	1.21
11-5-X-C2-3.0	141	116	1.22	114	1.24
North Carolina State University					
8-5-O-C2-2.5	140	80	1.75	85	1.64
8-8-O-C2-1.5	151	122	1.24	102	1.49
8-8-X-C2-1.5	152	182	0.84	127	1.20
11-5-O-C2-2.0	132	119	1.11	100	1.32
11-5-O-C3-2.0	151	119	1.27	121	1.24
11-5-X-C2-2.0	127	137	0.93	107	1.19
11-5-X-C3-2.0	155	137	1.13	135	1.15
11-8-O-C2-3.0	116	79	1.47	84	1.37
11-8-X-C2-3.0	128	123	1.04	116	1.11
			1.10	AVG.	1.29
			0.23	ST. DEV.	0.13
			0.21	COV	0.10
			1.75	MAX	1.64
			0.66	MIN	1.03

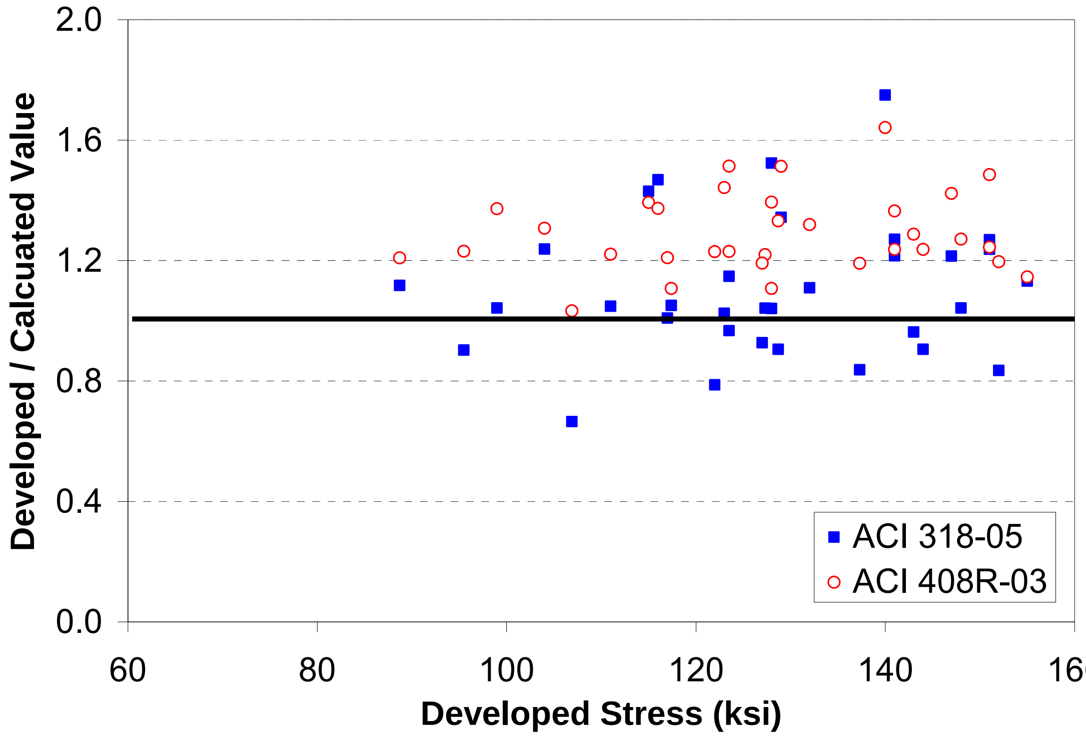

Figure 9: Distribution of developed/calculated values of confined splices

APPENDIX

Listed below are the reports or theses prepared by the different researchers of this cooperative research program. These documents can be obtained without charge from the Web sites as indicated. Those who are interested in more details of the research are encouraged to download the documents for review.

Briggs, M., Miller, S., Darwin, D., and Browning, J., "Bond Behavior of Grade 100 ASTM A 1035 Reinforcing Steel in Beam-Splice Specimens," SL Report 07-01, The University of Kansas Center for Research Inc., Lawrence, KS, 2007, 83 pp. http://www.mmfxsteel.com/techpub.shtml

Donnelly, K., "Behavior of Minimum Length Splices of High-Strength Reinforcement," Undergraduate Honors Thesis, University of Texas at Austin, Austin, TX, 2007, 27 pp.
http://www.engr.utexas.edu/research/fsel/FSEL_reports/Thesis/Donnelly,%20Kristen.pdf

Hosny, A., "Bond Behavior of High Performance Reinforcing Bars for Concrete Structures," M.Sc. Thesis, North Carolina State University, Raleigh, NC, 2007, 150 pp.
http://www2.lib.ncsu.edu/catalog/?Nty=1&N=0&Ntt=hosny&Ntk=Author

Hoyt, K., "Effect of Confinement and Gauging on the Performance of MMFX High Strength Reinforcing Bar Tension Lap Splices," M.Sc. Thesis, University of Texas at Austin, Austin, TX, 2007, 65 pp.
http://www.engr.utexas.edu/research/fsel/FSEL_reports/Thesis/Hoyt,%20Kathryn.pdf

Glass, G. M., “Performance of Tension Lap Splices with MMFX High Strength Reinforcing Bars,” M.Sc. Thesis, University of Texas at Austin, Austin, TX, 2007, 141 pp.
http://www.engr.utexas.edu/research/fsel/FSEL_reports/Thesis/Glass,%20Gregory.pdf

Seliem, H. M., “Behavior of Concrete Bridges Reinforced with High-Performance Steel Reinforcing Bars,” Ph.D. Dissertation, North Carolina State University, Raleigh, NC, 2007, 259 pp.
http://www2.lib.ncsu.edu/catalog/?Nty=1&N=0&Ntt=seliem&Ntk=Author

Seliem, H. M., Hosny, A., and Rizkalla, S., “Evaluation of Bond Characteristics of MMFX Steel,” Technical Report No. RD-07-02, Constructed Facilities Laboratory (CFL), North Carolina State University, 2007, 71 pp.
(http://www.mmfxsteel.com/techpub.shtml)