ABSTRACT

HOSNY, AMR. Bond Behavior of High Performance Reinforcing Bars for Concrete Structures. (Under the direction of Dr. Sami Rizkalla.)

Bond between the concrete and the reinforcing steel is a major factor affecting the performance of reinforced concrete structures. Advances in material science led to the production of High Performance Steel that has enhanced corrosion resistance and higher strength compared to conventional Grade 60 steel. Such material can lead to more economical design reducing the material requirements for a particular project and expanding its life span.

The objective of this research is to study the bond behavior of High Performance reinforcing bars for concrete structures and to evaluate the effect of different parameters believed to affect the bond characteristics. Twenty-two large scale reinforced concrete splice beams were constructed using No.8 and No.11 reinforcing bars, having different cross-sections with varying concrete compressive strengths and development lengths. The beams were tested using four point bending setup to provide a constant moment region over the splice zone. Test results indicate that stresses up 90 ksi can be achieved in the No.8 bars and up to 70 ksi in the No.11 bars without confinement; however, it is recommended to use transverse reinforcement to confine the High Performance bars in order to ensure ductility. These stresses can be evaluated at failure using a simple proposed equation. Test results were used to extend the current ACI Committee 408 equations to better predict the stresses in the High Performance Steel.
BOND BEHAVIOR OF HIGH PERFORMANCE REINFORCING BARS FOR CONCRETE STRUCTURES

By

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Dedication

To my mother

Who is the reason why I am everything I am.

Thank you.
Amr Hosny began his study of engineering in 1999 at Ain Shams University, Cairo, Egypt. In 2004, he graduated with honors and obtained his Bachelor of Science in Civil Engineering with a concentration in Structural Engineering. A year later, Amr enrolled in the Graduate Program at North Carolina State University to pursue a Master of Science degree in Civil Engineering with a concentration in Structures and Mechanics. Upon completion of his Master's, Amr intends to continue his research in pursuit of his Doctor of Philosophy degree at North Carolina State University.
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Chapter 1

Introduction

1.1 Background

Bond between the concrete and the reinforcing steel is a major factor in the performance of reinforced concrete structures. Initial research efforts have investigated the bond characteristics of conventional steel with concrete. Advances in material science have led to the production of High Performance reinforcing bars. These bars typically have an improved corrosion resistance and higher strength when compared to the conventional Grade 60 Steel. Using such bars can lead to a more economical design reducing the material requirements for a particular project and expanding its life span. Due to the non-linearity of the stress-strain relationship of the High Performance steel, the bond behavior between the concrete and the High Performance Steel has to be investigated including the effect of the different parameters believed to affect the bond characteristics. In addition, the applicability of the current code equations was evaluated for the case of using High Performance Steel as longitudinal reinforcement. The experimental program reported in this thesis is the first phase of an ongoing research investigating the bond behavior of High Performance Reinforcing Bars.
1.2 Objective

The primary objectives of the research presented in this thesis are:

1. Examine and identify the bond characteristics of High Performance reinforcing bars for concrete structures.

2. Evaluate the effect of different parameters affecting the bond behavior.

3. Investigate the applicability and accuracy of the code equations with High Strength Steel as longitudinal reinforcement for flexural members.
3.3 Scope

In order to investigate the bond behavior of the High Performance reinforcing bars, twenty two large scale splice beams have been constructed and tested at the Constructed Facilities Laboratory, at North Carolina State University. The splice specimens were tested in a four point bending test setup to develop a constant moment region at the spliced bars location. For the High Performance reinforcement, commercially available steel, know as MMFX Steel (Micro-Composite Multi-Structural Formable Steel), was used.

In addition to this introductory chapter, this thesis includes:

Chapter 2 gives a review of the relevant research conducted to this date on the bond between the concrete and the reinforcing steel.

Chapter 3 describes the experimental program, including the design of the test specimens, the construction of the specimens, the test setup, the instrumentation and the testing method.

Chapter 4 presents the test results obtained from the experimental program with the analysis of these results.

Chapter 5 summarizes the findings of this research and presents recommendations for future research in the field.

In addition to the main body of the thesis, two appendices are included to provide more details of the reinforcement of the test beams and the details of some of the test results.
Chapter 2

Background

2.1 Introduction

Bond between the concrete and the reinforcing steel plays a major role in the performance of reinforced concrete structures. This chapter briefly reviews the bond characteristics between the concrete and the reinforcing steel which affect in general the structural performance of the members. The bond consists mainly of three components:

- Chemical adhesion between the bars and the concrete.
- Frictional forces between the bars and the concrete due to the roughness of the surface of the bars in contact with the concrete.
- Mechanical anchorage or bearing of the ribs against the concrete surface.

It is important to note that the role of the bearing of the ribs against the concrete surface constitutes the major bond forces compared to the roles of the chemical adhesion and the frictional forces (ACI 408R-03).

2.2 Bond Behavior

The bearing behavior of the reinforcing steel on the concrete has been studied by many researchers over the years. It has been agreed that this behavior can be summarized as follows: (Orangun, Jirsa and Breen, 1977; Azizinamini, Chisala and Ghosh, 1995; Esfahani and Rangan, 1998)
A sector of a deformed bar embedded in concrete and subjected to axial tensile forces is shown in figure 2-1. This figure shows a free body diagram of the reinforcing bar at several load stages. In the beginning, when the axial load level is still low, the outermost lugs (the closest to the loading point) come in contact with the concrete as shown in Figure 2-1(a). Thus, these lugs exert a bearing force on the concrete. The horizontal component of this force produces bond stresses. (The horizontal component of the friction force is not shown in this figure but also adds to the bond strength.) Figure 2-1 also shows the horizontal bond stress distribution. As load increases, the bearing forces, exerted by the lugs, cause crushing of concrete in the vicinity of the lugs. This action allows the next adjacent lug to come in contact with concrete and participate in resisting the applied axial tension force as shown in Figure 2-1(b). The ACI building code assumes that at ultimate load, the bond stress distribution is uniform, which means that all the lugs bear against the concrete at the ultimate stage as shown in figure 2-1(c) and help resist the applied axial tensile force.
2.3 Factors Affecting Bond Characteristics

Many factors affect the bond between the reinforcing steel and the concrete. These factors can be distinguished under the following three categories:

- Structural characteristics
- Bar properties
- Concrete properties

2.3.1 Structural Characteristics

A brief discussion of some of the structural characteristics is included in the following section. These characteristics are: concrete cover and bar spacing, the...
bonded length of the bar, the degree of transverse reinforcement and the bar casting position.

2.3.1.1 Concrete Cover and Bar Spacing

As previously explained, the stress from a deformed bar is transferred to the concrete mainly by mechanical locking of the lugs with the surrounding concrete. The resultant force exerted by the lug on the concrete, $R$ is inclined at an angle $\beta$ to the axis of the bar as shown in figure 2-2. The radial component of these forces causes splitting of the surrounding concrete at failure.

![Figure 2-2: Bond forces on bar](image)

The behavior of the radial stresses, induced by the radial forces, on the surrounding concrete is similar to the water pressure acting against a thick walled cylinder having an inner diameter equal to the bar diameter and a thickness “c”. The thickness “c” is the smaller of (1) the clear bottom cover $c_b$ (2) half the bar clear spacing $c_{si}$ or the concrete side cover $c_{so}$ as shown in Figure 2-3a (please note that both the $c_{si}$ and the $c_{so}$ are referred to as $c_s$ in Figure 2-3). The capacity of the cylinder depends on the tensile strength of the concrete. With $c_b > c_s$, a horizontal split develops at the level of the bars, and is termed “side split failure”. When $c_s > c_b$, 
a “face-and-side split failure” forms with longitudinal cracking through the bottom
cover followed by splitting along the plane of the bars. When $c_s >> c_b$, a “V-notch
failure” forms with longitudinal cracking followed by inclined cracking. For the case of
lap splices where bars are side-by-side, the two cylinders to be considered for each
splice interact to form, in section, an oval ring, as shown in figure 2-3b. The failure
patterns are similar to those of single bars. (Orangun, Jirsa and Breen, 1977)

Figure 2-3: Failure patterns of anchored bars (Orangun, Jirsa and Breen, 1977)

According to the ACI committee 408 (ACI 408R-03), “the bond strength
increases as cover and bar spacing increase” but “this relationship, however, is not
linear, and the data shows that as the cover increases, the efficiency decreases”
(Canbay and Frosch, 2005). This can be explained due to the variation of the
distribution of the tensile stresses on the surrounding concrete from the spliced bar
as shown in figure 2-4.
Using the pipe analogy, as the inner diameter of the pipe increases (diameter of the reinforcing bar) for a constant wall thickness (the cover), the relative thickness of the wall of the pipe decreases, and thus the efficiency decreases. Canbay and Frosch also concluded that “the effective cover thickness can be related to the square root of the cover thickness-to-diameter of the spliced bar ratio $\sqrt{c/d_b}$.”

It is also worth mentioning that the cover plays a major role in the mode of failure of the beam, for instance, for a large cover and bar spacing, a pullout failure may occur. For a smaller cover and bar spacing, a splitting failure mostly occurs, as explained above, and it is the type expected to govern for most of the structural members. Pullout like failure can occur with some splitting if the member has significant transverse reinforcement to confine the anchored steel.

2.3.1.2 Bonded Length of the Bar

The increase in the development, bonded, or splice length will result in the increase in the bond strength or the bond capacity to a certain extent. This relationship was also found to be non linear, an increase in the bonded length by a
certain percentage induces an increase in bond strength but in a different percentage (Ferguson and Breen, 1965; Darwin et al., 1996b). “Test results indicate that doubling the splice length does not double the splice strength” (Canbay and Frosch, 2005). This can be explained by the nature of the bond stresses along the longitudinal reinforcing bars; as mentioned before, these stresses are assumed to be constant at ultimate but in reality they are not, the stresses are higher at both ends than at the center of the splice as demonstrated in Figure 2-5 and that explains the fact that the splitting cracks start at the end of the splice and propagate towards the center.

![Bond Stress Distribution](image)

Short Splice Length

Long Splice Length

Figure 2-5: Assumed bond stresses over the splice length (Canbay and Frosch, 2005)

From Figure 2-5 it is obvious that the assumption of a constant stress over the spliced length of the bars (ACI 318 2005) is more accurate for the short splices than it is for the long lap splices. It is also worth mentioning that “the splice length is relative” (Canbay and Frosch, 2005), meaning, that for different bar diameters the same splice length can be considered long for the smaller bar diameters and can be considered short for the larger ones.
To sum up, the relationship between the splice length and the splice strength was thought to be nearly linear, as indicated by the ACI committee 408 but further examination of the database of the ACI committee 408 by Canbay and Frosch (2005) indicated that this relationship is not linear and that the splice strength is proportional to the square root of the ratio of the splice length to the diameter of the reinforcing bar $\sqrt{\frac{l_d}{d_b}}$.

### 2.3.1.3 The Degree of Transverse Reinforcement

The ACI design philosophy is based on designing members to exhibit some level of ductility before failure. Previous studies have shown that the effect of transverse reinforcement confines the spliced bars and limits the progression of the splitting cracks. This confinement leads to the increase in the force required for the failure and can lead to a shorter requirement for the development or splice length. However, this is only valid up to a certain level of confinement, after that level the increase in the confinement becomes less effective providing no increase in bond strength. Although the ACI does not provide a minimum requirement for transverse reinforcement, it strongly encourages designers to include some in their design. It is also worth mentioning that increasing the amount of transverse reinforcement can change the mode of failure from splitting failure to pullout failure (Orangun, Jirsa and Breen, 1977; Azizinamini, Chisala and Ghosh, 1995; ACI 318-05; ACI 408R-03).

Many researchers have tried to evaluate the contribution of the confining steel on the bond strength as a term that would be added to the concrete contribution to
form the bond strength. In their research, Orangun, Jirsa and Breen (1977) have evaluated this contribution to be equal to:

$$
\frac{u_{tr}}{\sqrt{f'_c}} = \frac{1}{500} \left( \frac{A_{tr}f_{yt}}{s d_b} \right) \leq 3
$$

Where: $u_{tr}$ is the bond stress from the transverse steel, $A_{tr}$ is the area of transverse reinforcement normal to the plane of splitting through the anchored bars, $f_{yt}$ is the yield strength of the transverse reinforcement, $s$ is the spacing of the transverse reinforcement and $d_b$ is the bar diameter. It should be mentioned also that there is an upper limit of 3 to this equation, which is the value the authors found to be the upper limit after which the increase in transverse reinforcement will no longer be effective.

In a later effort done by Darwin et al. (1996a, 1996b), trying to represent and capture the effect of the transverse reinforcement on the bond strength, they found that this effect can be primarily represented by the equation:

$$
\frac{T_s}{f'_c^{3/4}} = \frac{N A_{tr}}{n}
$$

Where: $N$ is the number of transverse reinforcing bars crossing $l_d$, $A_{tr}$ is the area of each stirrup or tie crossing the potential plane of splitting adjacent to the reinforcement being developed or spliced and $n$ is the number of bars developed or spliced along the plane of splitting. Two points worth mentioning here: one is the absence of the yield strength of the transverse reinforcement from the equation. The investigators found that the transverse reinforcement rarely yields at the bond failure and it is the total area of the transverse steel that affects the increase in the bond strength. The other point is the normalization of the concrete compressive strength.
to the $\frac{1}{4}$ power instead of the square root finding that the quadratic root is a more accurate representative of the relationship. They also found in their study that an increase in the size of the spliced bars accompanied by an increase in the relative rib area (ratio of the projected rib area normal to the bar axis to the product of the nominal bar perimeter and the center-to-center rib spacing), increases the bond strength provided by the transverse reinforcement.

This equation was then modified to account for the effect of the relative rib area and the bar diameter using two parameters, namely $t_r$ and $t_d$, respectively. These parameters were obtained after the analysis of test results done by Darwin et al. (1996a, 1996b) and the relationships are as follows:

$$t_r = 9.6R_r + 0.28 \quad \text{and} \quad t_d = 0.72d_b + 0.28$$

After incorporating these parameters into the previous equation and after regression analysis for the available results, the authors concluded the effect of the transverse reinforcement on the bond strength of the splice to be:

$$\frac{T_s}{f_{c,\frac{1}{4}}} = 2226t_rt_d \frac{NA_{r,v}}{n} + 66$$

Furthermore, Zuo and Darwin (2000) analyzed more test results containing a wider scatter in the concrete compressive strength for the specimens in an effort to try to more accurately capture the effect of the concrete compressive strength on the contribution of the transverse steel to the bond strength. They concluded that $f_{c,\frac{1}{4}}$ to the power of $\frac{3}{4}$ is a more accurate representative of the effect of the concrete compressive strength than the $\frac{1}{4}$ power in the bond strength relationship:

$$\frac{T_s}{f_{c,\frac{3}{4}}} = 31.14t_rt_d \frac{NA_{r,v}}{n} + 3.99$$
On another note and in an effort to go back to the basics, and in an attempt to model bond behavior considering a physical modal of concrete tension cracking in the lap spliced region, Canbay and Frosch (2005) tried to establish an equation to evaluate the effect of confinement on bond strength. Knowing that multiplying the area of the transverse reinforcement by the exhibited stresses will give the forces carried by the transverse reinforcement, they proposed the following equation for the case of side-splitting:

\[ F_{\text{stirrup}} = \sum A_{\text{stirrup}} \sigma_{\text{stirrup}} = N_{st} N_I A_{st} \sigma_{st} \]

Where: \( N_{st} \) is the number of stirrups within the splice length; \( N_I \) is the number of stirrup legs; \( A_{st} \) is the area of stirrups; and \( \sigma_{st} \) is the stress of the stirrups. According to the authors, as the crack passages are different in the case of side-splitting and the case of face-splitting, there was a need to define an expression for each case. The expression for the face-splitting is given by:

\[ F_{\text{stirrup}} = N_{st} n A_{st} \sigma_{st} \]

Where: \( n \) is the number of bars being developed or spliced.

Canbay and Frosch (2005) also found that when there is a small amount of transverse reinforcement in the splice length, there is a noticeable scatter in the stresses of the stirrups over the plane of splitting but when the amount of transverse reinforcement increases over the splice length, the scatter decreases.

In the case of high strength reinforcing steel, Ferguson and Breen (1965) indicated also in their study that the stirrups increased the splice strength whether the amount of stirrups was minimal or for a heavily confined section. They also noted that the cracks were wider in the case where confining transverse reinforcement was present.
2.3.1.4 Bar Casting Position

The bar casting position plays an important role in the bond strength between the reinforcing steel and the concrete. It was found that as the depth of concrete below the bar increased, the bond strength decreased. This phenomenon can be explained due to the build up of bleed water around top cast bars and settlement of particles and aggregates in the concrete underneath. It was also found that the impact of a small top cover is larger than the effect of the casting position as the strength reduction factor becomes greater as the cover decreases.

The ACI Committee 408 also enforced the recommendations made by the ACI Committee 318 to increase the development length by 30% for the top cast bars.

2.3.2 Bar Properties

Bar properties have an effect on the bond strength between the bar and the surrounding concrete. Some of these properties, which include bar size, bar geometry, steel stress and yield strength as well as the bar surface condition, will be discussed in this section.

2.3.2.1 Bar Size

In general, bond strength increases as bar size increases for a given development length and for the same confinement level. However, to develop a certain bar strength, a longer development, or more precisely embedment, length is
needed for larger bar diameters, thus it is always favorable to use a large number of small bars than to use a small number of large bars under the condition that the spacing between the bars doesn’t become so small to the point of decreasing the bond strength.

Another consideration worth mentioning is that the bar size has an effect on the contribution of the transverse reinforcement to the bond strength; this can be explained by the fact that the slip of larger bar diameter will lead to higher strains, which means higher stresses, mobilized in the transverse reinforcement and thus better confinement. To sum up, as the bar size increases, the bond strength provided by the transverse reinforcement increases. (ACI Committee 408 2003)

2.3.2.2 Bar Geometry

The effect of bar geometry has been studied by many researchers over the years, and it has become gradually clearer over time. The researchers have concluded that the effect of bar geometry is mainly related to the ratio of the bearing area, which is the projected rib area normal to the bar axis, to the shearing area, which is the nominal bar perimeter multiplied by the center-to-center rib spacing, referred to as the relative rib area, $R_r$.

$$R_r = \frac{\text{Projected rib area normal to bar axis}}{\text{nominal bar perimeter} \times \text{center-to-center rib spacing}}$$

An increase in the relative rib area resulted in an increase in the initial part of the load slip curve. Test results have also shown that the degree of confinement has an effect on the increase in bond strength provided by the increase in relative rib
area; for bars not confined by transverse reinforcement the increase in relative rib area had a little effect on the bond strength, but, for bars confined by transverse reinforcement the effect of the increase of relative rib area had a considerable increase on the bond strength, and that explains the recommendations to implement the effect of the relative rib area coefficient, $t_r$, into the effect of the transverse reinforcement on the bond strength (Darwin and Graham, 1993; Darwin et al. 1996a).

2.3.2.3 Steel Strength and Yield Strength

It was believed previously that the bars that yielded before bond failure produced lower bond strength than bars that didn’t yield prior to such failure; therefore the design trend was set towards designing specimens that will not yield prior to failure. On the contrary, it was found that the bars without transverse reinforcement that yielded gave almost the same bond strengths as bars that didn’t yield. For the case of bars with transverse reinforcement, the bars that yielded averaged even a little higher than bars that didn’t yield (Darwin et al. 1996a; Zuo and Darwin, 2000).

It was observed that when high strength steel is used, the concrete adjacent to the bars will have greater strains and cracks when compared to when normal strength steel is used, and thus reducing the ultimate bond strength (ACI Committee 439, 1973). It was also observed for the case of high strength reinforcing steel with little or no yield plateau that an excess of flexural strength was exhibited. Furthermore, this excess of strength was exhibited in the lightly reinforced beams
more so than the heavily reinforced beams. This phenomenon can be explained due to the fact that the concrete of the beams with lightly reinforced cross section is subjected to low strains throughout the loading history which enables the steel to pick up additional stresses after the yield stress is reached and goes into the strain hardening region before the concrete ultimate strain is reached, failure is then attained due to crushing of concrete (Guralnick 1960). This observation was also confirmed by Sinha and Ferguson (1964) in their study where they concluded that not only were the beams reinforced by high strength steel in both tension and compression able to carry decreasing loads with increasing deflections after the concrete has crushed but also that their hinge characteristics were improved.

2.3.2.4 Bar Surface Condition

The bar surface conditions include the cleanliness of reinforcement, the presence or absence of rust from the bar surface and whether or not the bar is epoxy coated. The bar surface conditions have an effect on the bond strength as they affect the friction between the bar and the concrete and also affect the capability of the rib area to transfer the bond forces (ACI 408R-03).

2.3.3 Concrete Properties

Many of the concrete properties affect its bond with the reinforcing steel. A brief introduction of some selected parameters is discussed, including: compressive strength and aggregate type and quantity.
2.3.3.1 Concrete Compressive Strength

The effect of concrete compressive strength on the bond characteristics have been studied by many researchers. This effect is related to the square root of the compressive strength of the concrete in most of the equations describing the bond strength. It was found that the accuracy of such normalization is adequate up to compressive strength of 8000 psi then decreases as the compressive strength increases (Azizinamini, Chisala and Ghosh, 1995; Zuo and Darwin, 1998). The bearing action between the rib of the reinforcing steel and the high strength concrete is different than with normal strength concrete; as the bearing capacity increases more rapidly than the tensile strength preventing the crushing of the concrete in front of the ribs and thus reducing slip, this reduced slip results in fewer ribs transferring the load which increases the local tensile stresses and initiates the splitting failure in the concrete before achieving a uniform stress distribution along the splice or development length (Azizinamini, Chisala and Ghosh, 1995).

It was later observed that the quadratic root of the concrete compressive strength is a better representative of the concrete contribution to the bond strength equations than the square root (Darwin et al. 1996; Zuo and Darwin, 2000). As for the effect of the transverse reinforcement, it was found that the concrete compressive strength to the ¾ power is a good representative for the effect of the concrete compressive strength factor (Zuo and Darwin, 2000).
2.3.3.2 Aggregate Type and Quantity

The type of aggregate and its quantity affect the bond strength between the concrete and the reinforcing steel. For the case of absence of transverse reinforcement, it was observed that a higher strength coarse aggregate (basalt) will increase the effect of the concrete contribution on the bond characteristics than a weaker coarse aggregate (limestone). This observation is due to the fact that the higher strength coarse aggregate will increase the resistance to crack propagation which delays the splitting failure and increases the bond strength. It was also observed that the quantity of the aggregate did not have a significant effect on the bond strength.

As for the case of confining transverse reinforcement, it was observed that, both the aggregate type and quantity had an effect on the contribution of transverse reinforcement to the bond strength. The higher the strength of the coarse aggregate, the stronger the bond forces became. These observations can help explain the wide scatter of test results in the effect of the transverse reinforcement contribution on the bond strength compared to that of the concrete contribution (Darwin et al. 1996b; Zuo and Darwin, 2000; ACI 408R-03).

2.4 Evaluation of Bond Forces

Many researchers have tried to develop expressions to evaluate the bond strength of reinforcing bars. One of these researches is the work done by Orangun, Jirsa and Breen (1977). They proposed an equation to predict the average bond
stress at failure normalized to the square root of the concrete compressive strength $f'_{c}$, which is a measure of tensile strength, thus eliminating the effect of the variation of this parameter. Their equation was based on the test results of 62 specimens.

$$\frac{u_{cal}}{\sqrt{f'_{c}}} = 1.2 + \frac{3C}{d_{b}} + \frac{50d_{b}}{l_{d}}$$

The authors then compared their equation with the results from 9 studies of splice and development strength and found the predicted strengths to be very close with the results. The researchers also found that their expression became more conservative as the ratio between the transverse spacing of the reinforcing bars and the concrete cover, normalized to the bar diameter $C_{s}/(C_{b}d_{b})$ increased, where $C_{s}$ is equal to the minimum of one half of clear spacing of the longitudinal reinforcement or the side cover, $C_{b}$ is the cover and $d_{b}$ is the bar diameter.

A later study done by Darwin et al. (1992) found that there might have been some unintentional bias in the formulation of the previous equation due to the fact that the test results used in the regression analysis for the formulation of the previous equation contained some unfiltered results which might have affected the values of the coefficients obtained. For example, some of the specimens with larger bar diameter had a smaller lateral spacing than specimens with smaller bar diameter without accounting for that in the cover and thus an increase in the ratio of $C_{s}/C_{b}$ which leads to higher bond stresses as previously observed by Orangun et al. (1977).
In their research, Darwin et al. (1992) attempted to improve the previous equation by excluding the bias results and by analyzing the results with the ratio $C_s / C_b = 1$, separately from those with the ratio $C_s / C_b \neq 1$. Test results were also processed to evaluate the data with the cases where $C_s \geq C_b$ separately from the cases where the opposite happens. After the regression analyses of the test data included in that study, the authors introduced the following equation:

$$
\frac{A_s f_s}{\sqrt{f_c'}} = 6.67 l_a (C + 0.5d_b) \left(0.92 + 0.08 \frac{C_{\text{max}}}{C_{\text{min}}} \right) + 300 A_b
$$

A comparison of the results obtained from both equations showed that the results of the Darwin et al. (1992) equation provides a higher mean test/prediction ratio than Orangun, Jirsa and Breen's (1977) equation due to the conservative modifications introduced to the later best fit equation by Darwin et al. It was also found that the Darwin et al. equation gives a lower coefficient of variation than the Orangun, Jirsa and Breen equation thus showing improved accuracy. (Darwin et al. 1992)

In a later study, Darwin et al. (1996a, 1996b) concluded after processing the results of 133 bottom cast development and splice tests without confining transverse reinforcement and 166 tests with confining transverse reinforcement that the concrete contribution in the bond force can be expressed by:

$$
\frac{T_c}{f_c^{1/4}} = \frac{A_s f_s}{f_c^{1/4}} = \left[63 l_a (c_{\text{min}} + 0.5d_b) + 2130 A_b \right] \left(0.1 \frac{C_{\text{max}}}{C_{\text{min}}} + 0.9\right)
$$
Chapter 2
Background

It is worth mentioning here that in this equation the bond strength is represented related to the quadratic root instead of the square root of the compressive strength which was found to be more accurate. To better illustrate this observation a comparison of the results was undertaken by the authors of the aforementioned paper using the equation once with \( f_c \) to the ½ power and another time with \( f_c \) to the ¼ power; the results showed that the mean value from both equations was 1.00 and the coefficient of variation was 0.138 for the square root and 0.107 for the quadratic root, which illustrates the improved accuracy of the latter power. The previous equation is to be also modified to include the effect of the transverse reinforcement on the bond strength giving:

\[
\frac{T_b}{f_c^{1/4}} = \frac{T_c + T_s}{f_c^{1/4}} = A_b f_c^{1/4} = \left[ 63l_d \left( c_{\min} + 0.5d_b \right) + 2130A_b \right] \left( 0.1 \frac{c_{\max}}{c_{\min}} + 0.9 \right) + 2226t_d \frac{NA_{tr}}{n} + 66
\]

Where \( t_r \) is a term representing the effect of the relative rib area on \( T_s \) and,
\[ t_r = 9.6R_r + 0.28 \]
and,
\[ t_d = 0.78d_b + 0.28 \]

Using the same procedures followed by Darwin et al. (1996a, 1996b), and using the same database and adding some more results to it, Zuo and Darwin (2000) came up with the following equation to evaluate the effect of the concrete contribution to the bond strength:

\[
\frac{T_c}{f_c^{1/4}} = A_b f_c^{1/4} = \left[ 59.8l_d \left( c_{\min} + 0.5d_b \right) + 2350A_b \right] \left( 0.1 \frac{c_{\max}}{c_{\min}} + 0.9 \right)
\]
This equation is similar to that found by Darwin et al. (1996a, 1996b) with just the coefficient of \( l_d \) changing from 63 to 59.8 and the coefficient of \( A_b \) increasing from 2130 to 2350. This equation also showed a smaller coefficient of variation of 0.104 compared to 0.107 for the previous equation putting into consideration the increase in the number of the tests and thus the wider variation of the results (Zuo and Darwin, 2000). And as mentioned before, they also concluded that the ¾ power of the concrete compressive strength is a better representative than the ¼ power in the normalization of the contribution of the transverse reinforcement in the bond strength. Their equation finally becomes:

\[
\frac{T_b}{f_c^{1/4}} = \frac{T_c + T_s}{f_c^{1/4}} = A_b \frac{f_x}{f_c^{1/4}} = 59.8 l_d \left( c_{\text{min}} + 0.5 l_b \right) + 2350 A_b \left( 0.1 \frac{c_{\text{max}}}{c_{\text{min}}} + 0.9 \right) + 31.14 \frac{A_t}{n} + 3.99 f_c^{1/2}
\]

The ACI Committee 408 (ACI 408R-03) implemented minor changes to the previous equation and recommended the following:

\[
\frac{T_b}{f_c^{1/4}} = \frac{T_c + T_s}{f_c^{1/4}} = A_b \frac{f_x}{f_c^{1/4}} = 59.9 l_d \left( c_{\text{min}} + 0.5 l_b \right) + 2400 A_b \left( 0.1 \frac{c_{\text{max}}}{c_{\text{min}}} + 0.9 \right) + 30.88 \frac{A_t}{n} + 3 f_c^{1/2}
\]

Sensing that the bond strength equations have become very complicated, Canbay and Frosch (2005; 2006) attempted to model the bond behavior using a physical model of concrete tension cracking in the lap spliced region. They observed the crack patterns of the test specimens and based their equation on the simple formula that relates the force required to cause splitting and the surface area of the split times the stress in that area. Their equation, including the effect of the stirrups on the bond strength, is given as follows:
\[ f_b = \frac{F_{\text{splitting}} + F_{\text{stirrup}}}{nA_b \tan \beta} = \frac{l_d^* \left[ 2c_{st}^* + 2(n-1)c_{st}^* \right] 6\sqrt{f_c'} + N_{st} \sigma_{st} N_{tst}}{nA_b \tan \beta} \]

Where: \( f_b \) is the stress on the reinforcing bar, \( F_{\text{splitting}} \) is the radial force required to cause splitting, \( F_{\text{stirrup}} \) is the additional tension resistance across the splitting plane provided by the stirrups, \( A_b \) is the area of individual spliced bar, \( n \) is the number of bars being spliced or developed, \( \beta \) is a geometric relationship between the radial force and longitudinal bar force, \( l_d^* \) is the effective length which can be considered as the length of the splice where the stress distribution is uniform, \( c^* \) is the effective cover which can be also considered as the equivalent cover where the stress distribution is uniform.

\[
l_d^* = \frac{l_d}{\sqrt{l_d/d_b}} \left( \frac{1}{d_b} \right)^{2/3} \\
0.77 \leq \frac{c}{d_b} \leq 1.0
\]

The authors then solved their equation for \( l_d \) normalized to the bar diameter \( d_b \) and considering the minimum allowances by the ACI 318-05 for the concrete cover, bar spacing and amount of transverse reinforcement, to get an even more simplified version of the equation. The result was:

\[ \frac{l_d}{d_b} = \frac{f_y}{\sqrt{f_c'} 1 \times 10^6} \]

In this equation \( f_y \) is the yield strength of the reinforcing steel, and if used in ksi in stead of psi, the factor of \( 1 \times 10^6 \) in the denominator can be eliminated from the
equation. A comparison was then made between the results obtained from this equation and the equations proposed by the ACI 318-05 and the ACI 408R-03. It was found that all the equations can predict some values that are unconservative (ratio less than 1.0) for the unconfined test data, it was also found that the ACI 408R-03 equation with $\phi = 0.82$ provides the least number of unconservative results for unconfined test data and that when the $\phi$ factor is changed to 0.92, the percentage of unconservative results increases significantly. However, the ACI 408R-03 equation has the lowest standard deviation and coefficient of variation within the equations considered. As for the Canbay and Frosch (2006) equation, it performed well even though it was formulated for a minimum amount of transverse reinforcement and is not valid for the unconfined cases.

Although the percentage of unconservative results was high, the ACI 408R-03 equation gave the least standard deviation and coefficient of variation values among the other equations. As for the Canbay and Frosch equation, it gave a fairly reasonable distribution but the coefficient of variation was slightly larger than that for the ACI 408R-03 as was the average prediction ratio. While the development lengths calculated by this equation can be below those calculated by the current provisions for the smaller bars, the results are still conservative. Also an increase in safety was achieved using this equation relative to the use of the ACI 318-05 and the ACI 408R-03 design expressions.
2.5 The Micro-Composite Multi-Structural Formable Steel (MMFX)

The MMFX steel produced by MMFX Steel Corporation of America is believed to have high corrosion resistance compared to existing conventional steel in addition to its equal and in many cases superior properties of strength, toughness, energy absorption and formability. Research is being carried out across the United States and in other countries around the world to test this product and evaluate its properties.

Due to the microstructure of the conventional steel used in reinforced concrete, an electro-chemical reaction takes place between the Ferrite (anode) and the Iron-carbide (cathode) resulting in the formation of the Ferrous Oxides or rust. Therefore, if the elimination of the Carbides in the steel's micro-structure is possible, the chemical reaction causing the corrosion can be eliminated or minimized. The chemical composition and the production process of the MMFX steel has resulted in the fabrication of a virtually Carbide free steel. This composition has led to highly corrosion resistant steel which increases the service life of the steel and the structure.

As for the strength of the MMFX steel, it was found that it has greater yield strength than the conventional steel, which allows the use of less reinforcement area without losing the carrying capacity of the system or resulting in permanent deformations. It was also found that the steel has high ductile properties which provide safety against material failure due to the concentration of internal stresses. The MMFX steel was found to provide higher fatigue resistance, high toughness
allowing it to dissipate more energy before failure, and lower brittleness which allows the steel to retain its loading capacities in a wider range of temperatures.

All these improved properties have rendered the MMFX steel slightly more expensive when compared to the conventional steel, but that is only the case when the comparison is made with only the cost of rebar for both materials. When labor cost and the service life of the structures are taken into consideration, the cost of the MMFX steel becomes very competitive with the conventional steel. (MMFX Steel product bulletin)

The bond behavior of the MMFX steel is studied in this experimental research and the effect of different parameters affecting the bond is evaluated. In addition, a comparison of the test results with the available code equations is provided to evaluate the accuracy and applicability of such equations in evaluating the bond strength between the MMFX reinforcing bars and the concrete.
Chapter 3

Experimental Program

3.1 Introduction

This chapter describes the experimental program completed at the Constructed Facilities Laboratory, North Carolina State University, Raleigh, NC, to investigate the bond characteristics of High Performance reinforcing bars for concrete structures.

The study investigated the behavior of beam splice specimens tested to evaluate the effect of some selected parameters believed to be the most effective on the bond strength: Concrete cover, concrete compressive strength, bar size and confinement level. A collective test matrix for a comprehensive testing program considering the above parameters is given in Table 3-1. The first phase of this program reported in the thesis is highlighted in this table.
Table 3-1: Testing program:

<table>
<thead>
<tr>
<th>$f'_c$</th>
<th>$d_b$</th>
<th>Phase I</th>
<th>Phase II</th>
<th>Phase III</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Cover</td>
<td>Cover</td>
<td>Cover</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$3/4''$</td>
<td>$2d_b$</td>
<td>$3d_b$</td>
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<tr>
<td>#5</td>
<td></td>
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</tr>
</tbody>
</table>

Total: 22 22 22
3.2 Test Specimens

The experimental program consisted of 22 large scale reinforced concrete splice beams divided into two main categories according to the diameter of the longitudinal reinforcing bar. Within each category, two different concrete strengths were used. For each concrete strength, two different splice lengths were used as demonstrated in Figure 3-1.

The twenty two beams tested were designed to achieve select levels of stresses in the tension steel of 80, 100, 120 and 140 ksi for a specified splice length using different levels of confinement. The beams were designed with nominal ultimate flexural capacity assuming a stress level in the tension steel higher than 160 ksi. For simplification of the construction procedure, the beams were kept at a constant length of 23 feet. As for the dimensions, the width was selected to provide equal side and bottom covers as well as spacing between the longitudinal steel equal to double the value of the cover. The height was kept constant as much as possible for the ease of fabrication. Figure 3-2 shows a typical cross section of the
beams with and without confining transverse reinforcement followed by Table 3-2 and Table 3-3 showing the dimensions and reinforcement details of the test matrix.

Figure 3-2: Typical cross section of the beams
Table 3-2: Design of test beams

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>$d_b$</th>
<th>$f'_c$</th>
<th>Cover in.</th>
<th>Section “b x h” in.</th>
<th>Stress in Steel</th>
<th>Moment Capacity kip-ft</th>
<th>Applied Load kips</th>
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<tr>
<td>8-5-O-C0</td>
<td>8</td>
<td>5</td>
<td>2.5</td>
<td>14 x 24</td>
<td>162 40</td>
<td>400</td>
<td>67</td>
</tr>
<tr>
<td>8-5-O-C1</td>
<td>8</td>
<td>8</td>
<td>1.5</td>
<td>10 x 24</td>
<td>168 55</td>
<td>445</td>
<td>74</td>
</tr>
<tr>
<td>8-5-X-C0</td>
<td>11</td>
<td>5</td>
<td>2.0</td>
<td>14 x 36</td>
<td>165 62</td>
<td>1305</td>
<td>218</td>
</tr>
<tr>
<td>8-5-X-C1</td>
<td>11</td>
<td>8</td>
<td>3.0</td>
<td>18 x 24</td>
<td>161 23</td>
<td>754</td>
<td>126</td>
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<tr>
<td>8-8-O-C0</td>
<td>8</td>
<td>5</td>
<td>2.5</td>
<td>14 x 24</td>
<td>162 40</td>
<td>400</td>
<td>67</td>
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<tr>
<td>8-8-O-C1</td>
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<td>1.5</td>
<td>10 x 24</td>
<td>168 55</td>
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<td>8-8-X-C0</td>
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<td>161 23</td>
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<tr>
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<td>5</td>
<td>2.0</td>
<td>14 x 36</td>
<td>165 62</td>
<td>1305</td>
<td>218</td>
</tr>
<tr>
<td>11-5-O-C1</td>
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<td>3.0</td>
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<td>161 23</td>
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<td>8</td>
<td>3.0</td>
<td>18 x 24</td>
<td>161 23</td>
<td>754</td>
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### Table 3-3: Reinforcement details of test beams

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<th>Beam ID</th>
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<th>Section &quot;b x h&quot;</th>
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<th>Development length</th>
<th>Compression Steel</th>
<th>Stirrups Spacing</th>
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<tr>
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<td>11</td>
<td>14 x 24</td>
<td>2</td>
<td>91</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8-8-X-C1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>8.0</td>
</tr>
<tr>
<td>8-8-X-C2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4.0</td>
</tr>
<tr>
<td>11-5-O-C0</td>
<td></td>
<td>14 x 36</td>
<td>2</td>
<td>69</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11-5-O-C1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6.5</td>
</tr>
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<tr>
<td>11-5-X-C0</td>
<td>11</td>
<td>18 x 24</td>
<td>3</td>
<td>43</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11-5-X-C1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5.5</td>
</tr>
<tr>
<td>11-5-X-C2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2.5</td>
</tr>
<tr>
<td>11-8-O-C0</td>
<td></td>
<td>14 x 36</td>
<td>2</td>
<td>69</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11-8-O-C1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5.5</td>
</tr>
<tr>
<td>11-8-O-C2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2.5</td>
</tr>
<tr>
<td>11-8-X-C0</td>
<td>11</td>
<td>18 x 24</td>
<td>3</td>
<td>57</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11-8-X-C1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7.0</td>
</tr>
<tr>
<td>11-8-X-C2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.5</td>
</tr>
</tbody>
</table>
The beams, shown in Tables 3-2 and 3-3, are identified according to four parameters: the first number is either “8 or 11” to indicate the size of the longitudinal reinforcement, the second number indicates the targeted concrete compressive strength $f_{c}^\prime$ in ksi and this number is going to be either “5 or 8” for concrete compressive strengths of 5000 psi and 8000 psi, respectively. The third letter, “O or X”, is the selected development length to develop a stress level of 80 ksi or 100 ksi without confinement, respectively. The last parameter is the confinement level in the splice zone; “C0” indicates that there is no confining transverse reinforcement within the splice length, “C1 and C2” are the first and second confinement levels, respectively.

Table 3-2 and Table 3-3 indicate that the beams are divided into two main categories according to the diameter of the reinforcing bars. For each bar size, the beams were divided into two main groups with different targeted concrete compressive strengths. Within each of these groups, two sub groups with different development lengths were formed. Within each group, the concrete compressive strength $f_{c}^\prime$, the dimensions and the concrete cover are kept constant to isolate the effect of these parameters and evaluate the effect the development length and confinement levels separately. Within each sub group, the development length is the same and the confinement level is the only variable. The development length for each sub group was chosen to develop a selected stress level; for the beams with “O” in their identification, the targeted stress level to be developed with no confining transverse reinforcement is 80 Ksi, for the first confinement level the targeted stress is 100 Ksi and 120 ksi for the second level of confinement. As for the beams with “X”
in their identification, the targeted stress levels are 100 Ksi for the beams without confinement, 120 ksi and 140 ksi for the first and second confinement levels respectively.

The equation used to calculate the development length is equation (4-11a) of the ACI 408R-03 draft report given below

\[
\frac{l_d}{d_b} = \frac{\left(\frac{f_y}{f_c^{1/4}} - 2400\omega\right)\alpha\beta\lambda}{76.3\left(\frac{c\omega + k\omega}{d_b}\right)}
\]

Where;

\(l_d\) = The development length, in.;
\(d_b\) = The diameter of the bar, in.;
\(f_y\) = The yield strength of the reinforcing steel which is going to be substituted by \(f_s\), the targeted stress in the reinforcing bar, psi;
\(f_c\) = The concrete compressive strength, psi;
\(\alpha\) = The reinforcement bar location factor taken as unity as all bars are bottom cast;
\(\beta\) = The coating factor, taken as unity as all bars are uncoated;
\(\lambda\) = The lightweight concrete factor, taken as unity;
\(c\) = The concrete cover, in;
\(\omega\) = A factor that depends on the minimum and maximum concrete cover;

\[
\omega = 0.1 \frac{c_{\text{max}}}{c_{\text{min}}} + 0.9 \leq 1.25
\]
\[ k_{tr} = \text{The transverse reinforcement index; } \]
\[ k_{tr} = \left( \frac{0.52 t_r d_t A_{tr}}{sn} \right)^{0.5} \]

\[ t_r = \text{A term representing the effect of the relative rib area on the contribution } \]
\[ \text{of the transverse reinforcement on the bond strength; } \]
\[ t_r = 9.6 R_r + 0.28 \leq 1.72 \]

\[ R_r = \text{Relative rib area. For MMFX reinforcement } R_r \text{ has an average value of } \]
\[ 0.0913; \]

\[ t_d = \text{A term representing the effect of the bar diameter on the contribution of } \]
\[ \text{the transverse reinforcement on the bond strength; } \]
\[ t_d = 0.78 d_p + 0.22 \]

\[ s = \text{The spacing of the transverse reinforcement in the splice zone, in.; } \]

\[ n = \text{The number of bars being developed or spliced; } \]

The Relative rib area \( R_r \) influences the transverse reinforcement index. For conventional bars, the average value of \( R_r \) is 0.0727. Reports by previous researchers indicate that the relative rib area for MMFX is higher than for conventional steel. For instance, El-Agroudy (2003) found that \( R_r \) value for No.6 and No.8 bars is 0.1008 while Darwin (2005) reported that the typical value of \( R_r \) for No.6 MMFX bars is 0.0892. For the calculations of the transverse reinforcement index in this research, a measured value of 0.0913 is used for the relative rib area for bar sizes 8 and 11.
For the calculations of the nominal capacities of the beams, the stress-strain relationship proposed by Hognestad shown Figure 3-3 was used for concrete and the stress-strain relationship shown in Figure 3-4 was used for the MMFX steel:

Hognestad equations are as follow:

\[
E_c = 1.8 \times 10^6 + 460 f_c',
\]

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

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

For the parabola

\[
a_1 = \frac{\varepsilon_o}{\varepsilon_{cu}} c; a_2 = \frac{\varepsilon_{cu} - \varepsilon_o}{\varepsilon_{cu}} c
\]
No chairs were used to support the steel cage at the locations of the splices to eliminate any effect these chairs may have on the bond between the reinforcing bars and the concrete.

In addition to the confining steel along the splice length with designated spacing “S1” in table 3-3, transverse reinforcement was added outside the splice length with spacing “S2” as shown in Figure 3-5 to prevent premature failure due to shear. Furthermore, for the beams with no confinement along the splice length, U shaped stirrups were distributed along the compression reinforcement in the splice zone to prevent buckling of the compression reinforcement. The longitudinal top and bottom reinforcement are MMFX steel and the transverse reinforcements are conventional Grade 60 steel.
The complete cross sections of the beams tested in this experimental program are shown in Appendix A.
3.3 Construction of the Specimens

The test specimens were constructed at the Constructed Facilities Lab. A specially designed concrete casting bed was erected for this project and sheets of Plywood were placed on top of the concrete bed to provide the bottom part of the forms as shown in Figure 3-6.

![Casting Bed with the plywood placed on top](image)

Wooden forms were made out of Plywood and the steel cages were placed on the bed in the orientation that the spliced bars will be bottom cast to eliminate the effect of the casting position on the bond strength. Before the tests, the specimens were then rotated inside the lab to ease the observation of the cracks and their progress.

Forms were treated with form release agent to facilitate their stripping and once the steel cages were in place on the casting bed, the forms were put together in preparation for the casting procedure as shown in figure 3-7.
Chapter 3                                                                                   Experimental Program

Figure 3-7: the construction process of beams 8-8-X-C0 & C1

The concrete was supplied by a ready mix concrete local supplier and cast into the forms. An electric vibrator was used to consolidate the concrete before the screeding of the surface. The beams were then left on the casting bed and wet burlap and plastic sheets were left on the beams for three days to give the concrete a chance to cure. A week later the beams were removed from the bed and stacked together until the day of testing as shown in figure 3-8.

Figure 3-8: the beams reinforced with # 8 bars
3.4 Material Properties

In order to accurately evaluate the bond strength between the concrete and the reinforcing steel, material properties had to be determined. Specimens from the reinforcing steel were tested to evaluate the MMFX stress-strain properties and concrete cylinders were made from the concrete and tested as well.

3.4.1 Reinforcing Steel

The actual stress-strain characteristics of the reinforcing steel used in the beam specimens were determined based on an ASTM Standard test using an MTS universal tension-compression testing machine with hydraulic grips and 200 kips capacity. Test load and stroke from the testing machine were monitored and recorded and the strain reading was recorded from an MTS extensometer attached to the bar. Two samples of the No.8 MMFX bars were taken from the supply used to construct the splice beams.

Each specimen was subjected to gradually increasing uniaxial load until failure took place. Load was applied at a rate of 0.001 inches displacement per second. When the applied load necessary to produce additional strain began to decrease, the test was paused and the MTS extensometer was removed to prevent damage at rupture. The test was then continued until fracture of the specimen. Figure 3-9 shows the MTS extensometer attached to the bar during the tension test.
The stress-strain relationships obtained from the two specimens were identical as shown in figure 3-10. The best fit equation from these two curves is given by 

$$f_s = 159 (1 - e^{-248\varepsilon})$$

This equation is different from the equation proposed by the University of Texas where 

$$f_s = 156 (1 - e^{-220\varepsilon})$$

and since the difference in the results from the two equations is negligible as shown in figure 3-11, the later equation is used in this research for consistency.

The number 11 bars were not tested due to the limited capacity of the MTS machine.
The equation used for the stress-strain relationship of the No.11 bars was supplied by the University of Texas as:

\[ f_s = 162(1 - e^{-235\varepsilon}) \]

Figure 3-10: Stress-Strain relationship obtained from the tension tests

Figure 3-11: Comparison of the two equations with the test results
3.4.2 Concrete

The concrete was supplied by a local ready-mix company. Each sub group was cast at a time to eliminate the effect of the difference in compressive strength on the bond characteristics. From the experimental program, it can be seen that two grades of concrete strengths were used, 5000 psi and 8000 psi. A sample mix design for each of these grades is given in table 3-4.

Table 3-4: Sample of concrete mixes of test beams

<table>
<thead>
<tr>
<th>$f'_c$</th>
<th>w/c</th>
<th>Cement (lb/yd³)</th>
<th>Water (lb/yd³)</th>
<th>Sand (lb/yd³)</th>
<th>Stone (lb/yd³)</th>
<th>Flyash (lb/yd³)</th>
<th>Water Retarder (oz/yd³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5000</td>
<td>0.43</td>
<td>528</td>
<td>287</td>
<td>1331</td>
<td>1700</td>
<td>132</td>
<td>66</td>
</tr>
<tr>
<td>8000</td>
<td>0.36</td>
<td>680</td>
<td>306</td>
<td>1260</td>
<td>1550</td>
<td>170</td>
<td>68</td>
</tr>
</tbody>
</table>

Type I Portland cement was used in the preparation of the concrete. The maximum specified aggregate size was 3/8 in. to ensure good flow of the concrete around the steel cage and eliminate formation of any honey combing. A slump test was also performed to ensure the workability of the concrete and the results varied from 4 in. to 6.5 in.

Fifteen 4 in. x 8 in. cylinders were made from each batch to be tested after three days, seven days and twenty eight days after casting as well as on the days of testing of the splice beams. The concrete compressive strength for all tested beams is summarized in Table 3-5.
Table 3-5: Summary of concrete compressive strength of test beams at the day of testing

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Cylinder #1 psi.</th>
<th>Cylinder #2 psi.</th>
<th>Cylinder #3 psi.</th>
<th>Average psi.</th>
</tr>
</thead>
<tbody>
<tr>
<td>8-5-O-C0</td>
<td>5681</td>
<td>6257</td>
<td>6057</td>
<td>6015</td>
</tr>
<tr>
<td>8-5-O-C1</td>
<td>5935</td>
<td>5845</td>
<td>5710</td>
<td>5817</td>
</tr>
<tr>
<td>8-5-X-C0</td>
<td>8359</td>
<td>8242</td>
<td>8615</td>
<td>8400</td>
</tr>
<tr>
<td>8-8-O-C0</td>
<td>9934</td>
<td>10302</td>
<td>10364</td>
<td>10200</td>
</tr>
<tr>
<td>8-8-X-C0</td>
<td>5398</td>
<td>5362</td>
<td>5274</td>
<td>5344</td>
</tr>
<tr>
<td>8-8-O-C1</td>
<td>4205</td>
<td>3879</td>
<td>4090</td>
<td>4058</td>
</tr>
<tr>
<td>8-8-X-C1</td>
<td>4205</td>
<td>3879</td>
<td>4090</td>
<td>4058</td>
</tr>
<tr>
<td>11-5-O-C0</td>
<td>5938</td>
<td>5825</td>
<td>6300</td>
<td>6070</td>
</tr>
<tr>
<td>11-5-O-C1</td>
<td>8124</td>
<td>8638</td>
<td>8388</td>
<td>8383</td>
</tr>
<tr>
<td>11-5-X-C0</td>
<td>5398</td>
<td>5362</td>
<td>5274</td>
<td>5344</td>
</tr>
<tr>
<td>11-5-X-C1</td>
<td>4205</td>
<td>3879</td>
<td>4090</td>
<td>4058</td>
</tr>
<tr>
<td>11-5-X-C2</td>
<td>5938</td>
<td>5825</td>
<td>6300</td>
<td>6070</td>
</tr>
<tr>
<td></td>
<td>8124</td>
<td>8638</td>
<td>8388</td>
<td>8383</td>
</tr>
</tbody>
</table>
3.5 Test Setup

All beams were tested using four point bending configuration to develop a constant moment region at the spliced bars location. The length of the beams was kept constant at 23 feet to ease the construction as mentioned before. This constant length led to using the same test setup for all beams as shown in figure 3-12. The test setup allowed a constant moment region of 9 feet, where the splice length is located and two shear spans of length 6 feet each. The beams were supported at both ends using steel beams restrained to the floor. Two 150 kips load cells were placed at each end to measure the reactions at the supports. The loads were applied using four hydraulic jacks, two at each location, with a capacity of 120 kips each.

![Figure 3-12: Actual test setup](image)
3.6 Instrumentation

A total of four electrical resistance strain gages, six PI gages, six string pots and two load cells were used to measure the strains, the deflections and the loads on the beams respectively. These instruments were all connected to a Data acquisition system to record the data.

The four electrical resistance strain gages were attached to the longitudinal reinforcing bars immediately outside the splice zone to measure the strains in the spliced bars.

As for the concrete strain, six PI gages with a 100 mm. gage length were mounted on the top and the bottom of each beam along its center line at the ends of the splices and at mid-span. These PI gages were used to determine the crack width at these locations using the number of cracks crossing the gage length that were visually recorded during the test. In addition to the PI gages, a crack comparator was used to manually measure the crack width at different load levels. Figure 3-13 shows a sample of the PI gage distribution on the test beam.

![Location of PI Gages](image-url)

Figure 3-13: typical PI gage distribution along the beam
To measure the deflections, six string pots were used. Two were placed at both ends of the beam, two at each loading location and the remaining two were placed side-by-side along the center line of the beam. The total deflections reported are the relative deflections between the measured deflections at the ends of the beams and those measured at mid-span. The string pots attached to the loading points were used to indicate the stroke length of the hydraulic jack to give an indication of the remaining available stroke.
3.7 Test Procedure

The load was applied using hydraulic jacks that were manually controlled. The loading was paused at different load levels to visually inspect the beam, the crack pattern and determine the crack width using crack comparators. The readings from the instruments along with the readings of the load cells were captured by the data acquisition system at a rate of 1 scan/second. The test duration varied between 20 minutes for beam 11-8-O-C0 and 59 minutes for beam 8-5-X-C2.
Chapter 4

Test Results and Analysis

4.1 Overview

The experimental program consisted of 22 large scale reinforced concrete splice beams divided into two main categories according to the diameter of the longitudinal reinforcing bar. Within each category, two different concrete strengths were used. For each of these strengths, two different splice lengths were used as described in Figure 4-1.

Figure 4-1: Experimental program chart

The beams within each group are identical except in the amount of confining transverse reinforcement. Table 4-1 gives details of the reinforcement for each group tested in the experimental program.
Table 4-1: Reinforcement details of the experimental program

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Splice Bar Size</th>
<th>f_c'</th>
<th>Cover</th>
<th>Splice Length</th>
<th>Confinement Level along the splice length</th>
<th>Target Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>8-5-O-C0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Unconfined</td>
<td>80</td>
</tr>
<tr>
<td>Group 1</td>
<td>8-5-O-C1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>First Level</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>8-5-O-C2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Second Level</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td>8-5-X-C0</td>
<td></td>
<td>8</td>
<td>5</td>
<td>31</td>
<td>Unconfined</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>8-5-X-C1</td>
<td></td>
<td>8</td>
<td>2.5</td>
<td>41</td>
<td>First Level</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td>8-5-X-C2</td>
<td></td>
<td>8</td>
<td>2.5</td>
<td></td>
<td>Second Level</td>
<td>140</td>
</tr>
<tr>
<td>Group 2</td>
<td>8-8-O-C0</td>
<td></td>
<td>8</td>
<td>1.5</td>
<td>40</td>
<td>Unconfined</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>8-8-O-C1</td>
<td></td>
<td>8</td>
<td>1.5</td>
<td></td>
<td>First Level</td>
<td>100</td>
</tr>
<tr>
<td>Group 3</td>
<td>8-8-X-C0</td>
<td></td>
<td>8</td>
<td>2.0</td>
<td>54</td>
<td>Unconfined</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>8-8-X-C1</td>
<td></td>
<td>8</td>
<td>2.0</td>
<td></td>
<td>First Level</td>
<td>120</td>
</tr>
<tr>
<td>Group 4</td>
<td>11-5-O-C0</td>
<td></td>
<td>11</td>
<td>5</td>
<td>69</td>
<td>Unconfined</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>11-5-O-C1</td>
<td></td>
<td>11</td>
<td>5</td>
<td></td>
<td>First Level</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>11-5-O-C2</td>
<td></td>
<td>11</td>
<td>5</td>
<td></td>
<td>Second Level</td>
<td>120</td>
</tr>
<tr>
<td>Group 5</td>
<td>11-5-X-C0</td>
<td></td>
<td>11</td>
<td>5</td>
<td>91</td>
<td>Unconfined</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>11-5-X-C1</td>
<td></td>
<td>11</td>
<td>5</td>
<td></td>
<td>First Level</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td>11-5-X-C2</td>
<td></td>
<td>11</td>
<td>5</td>
<td></td>
<td>Second Level</td>
<td>140</td>
</tr>
<tr>
<td>Group 6</td>
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<td>43</td>
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<td>80</td>
</tr>
<tr>
<td></td>
<td>11-8-O-C1</td>
<td></td>
<td>8</td>
<td>3.0</td>
<td></td>
<td>First Level</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>11-8-O-C2</td>
<td></td>
<td>8</td>
<td>3.0</td>
<td></td>
<td>Second Level</td>
<td>120</td>
</tr>
<tr>
<td>Group 7</td>
<td>11-8-X-C0</td>
<td></td>
<td>8</td>
<td>3.0</td>
<td>57</td>
<td>Unconfined</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>11-8-X-C1</td>
<td></td>
<td>8</td>
<td>3.0</td>
<td></td>
<td>First Level</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td>11-8-X-C2</td>
<td></td>
<td>8</td>
<td>3.0</td>
<td></td>
<td>Second Level</td>
<td>140</td>
</tr>
<tr>
<td>Group 8</td>
<td>11-8-O-C0</td>
<td></td>
<td>8</td>
<td>3.0</td>
<td>57</td>
<td>Unconfined</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>11-8-O-C1</td>
<td></td>
<td>8</td>
<td>3.0</td>
<td></td>
<td>First Level</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td>11-8-O-C2</td>
<td></td>
<td>8</td>
<td>3.0</td>
<td></td>
<td>Second Level</td>
<td>140</td>
</tr>
</tbody>
</table>
4.2 Test Results

This section describes the observed mode of failure, measured stresses in the bars at failure, overall deflection as well as the crack width and pattern.

4.2.1 Mode of Failure

Failure of the spliced beams was nearly identical. All beams without confining transverse reinforcement along the splice length failed suddenly after the initiation of the splitting cracks without warning or propagation of the cracks accompanied by loss of the concrete cover over the entire splice length as shown in Figure 4-2. The failure mode was greatly affected by the presence of confinement, therefore, typical mode of failure for the first group of beams representing the beams reinforced with No.8 bars, and the eighth group of beams representing the beams reinforced with No.11 bars are presented in this section. A typical mode of failure by sudden loss of the concrete cover for the beams without confining transverse reinforcement is shown in Figure 4-2 for beam 8-5-O-C0 from the first group of beams and in Figure 4-3 for beam 11-8-O-C0 from the eighth group of beams.
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Figure 4-2: Typical Splitting Failure for the beams reinforced with No.8 bars without confining transverse reinforcement

Figure 4-3: Typical Splitting Failure for the beams reinforced with No.11 bars without confining transverse reinforcement
Failure of the eight beams tested with the first level of confinement, C1, was also due to splitting of the concrete cover with the exception of beam 8-5-X-C1 which failed due to crushing of the concrete in the compression zone. The splitting failure of the beams with the first level of confinement was more ductile and allowed propagation of the splitting cracks prior to failure. Presence of the transverse reinforcement restrained propagation and widening of the splitting cracks. Figure 4-4 and Figure 4-5 show the typical splitting failure of the beams with the first level of confinement for beams with No.8 and No.11 bars sizes, respectively. From these figures, the loss of the concrete cover along the length of the splice accompanied with the presence of uniformly distributed flexural cracks in comparison to the two major cracks at the end of the splice length for unconfined beams.

Figure 4-4: Typical Splitting Failure for the beams reinforced with No.8 bars with the first level of confinement
Failure of the six beams tested with the second level of confining transverse reinforcement, C2, was mainly due to crushing of the concrete in the compression zone followed by loss of the concrete cover as shown in Figure 4-6 and Figure 4-7. Only two beams from this category failed due to splitting similar to the beams with the first level of confinement. For the flexural failure, the splitting cracks started to propagate over the splice length; however, the presence of the heavy transverse reinforcement provided sufficient confinement and prevented the splitting failure. Figure 4-6 shows the typical flexure failure for beam 8-5-O-C2 from the first group reinforced with No.8 bars with the second level of confining transverse reinforcement and Figure 4-7 shows the splitting failure for beam 11-8-O-C2 from the eighth group reinforced with No.11 bars.
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Figure 4-6: Typical Flexure Failure of the beams with the second level of confinement

Figure 4-7: Typical splitting failure for the beams with the second level of confinement
Table 4-2 summarizes the mode of failure of the beams tested in the experimental program. Failure modes for all tested beams in this program are given in appendix B part 1.
Table 4-2: Summary of the mode of failure for the tested beams

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Mode of failure</th>
<th>Failure load</th>
<th>Stress in the spliced bars at failure from cracked section analyses</th>
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<tr>
<td></td>
<td></td>
<td></td>
<td>Kips</td>
<td>Ksi</td>
</tr>
<tr>
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<td>8-5-O-C0</td>
<td>Splitting</td>
<td>40.0</td>
<td>96</td>
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<tr>
<td></td>
<td>8-5-O-C1</td>
<td>Splitting</td>
<td>58.6</td>
<td>140</td>
</tr>
<tr>
<td></td>
<td>8-5-O-C2</td>
<td>Flexure</td>
<td>65.1</td>
<td>152</td>
</tr>
<tr>
<td>Group 2</td>
<td>8-5-X-C0</td>
<td>Splitting</td>
<td>45.8</td>
<td>110</td>
</tr>
<tr>
<td></td>
<td>8-5-X-C1</td>
<td>Flexure</td>
<td>64.1</td>
<td>152</td>
</tr>
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<td></td>
<td>8-5-X-C2</td>
<td>Flexure</td>
<td>63.9</td>
<td>152</td>
</tr>
<tr>
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<td>39.9</td>
<td>91</td>
</tr>
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<td>8-8-O-C1</td>
<td>Splitting</td>
<td>67.1</td>
<td>151</td>
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<td>Splitting</td>
<td>67.6</td>
<td>151</td>
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<td>Splitting</td>
<td>199.0</td>
<td>151</td>
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<td>Splitting</td>
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<td>Flexure</td>
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<td>Flexure</td>
<td>113.0</td>
<td>157</td>
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4.2.2 Stresses in the Spliced Bars

The stresses in the spliced bars were evaluated based on the measured strains from the strain gages attached to the longitudinal reinforcement located immediately at the end of the splice zone. These measured strains were used to evaluate the stresses based on the measured stress-strain relationship for the No.8 and No.11 bars of the MMFX reinforcing steel and modeled as follows:

\[
\sigma_s = 156(1 - e^{-220\varepsilon}) \\
\sigma_s = 162(1 - e^{-235\varepsilon})
\]

for the #8 bars

for the #11 bars

Furthermore, the stresses in the bars at failure were evaluated using cracked section flexural analyses of the tested beams using the measured reactions of the load cells. The non-linearity of the stress-strain relationship of the MMFX Steel was considered in the analyses to determine the resulting stresses. In addition, moment-curvature analyses were calculated for each cross section of the tested beams in this experimental program. All three values were compared together to verify the accuracy of the reported results. Table 4-3 shows the measured stresses for the tested beams. From this table it can be seen that adding confining transverse reinforcement increases the measured stresses in the spliced bars significantly. For example, in group 1, the measured stresses in the longitudinal reinforcement for the beam without confinement were 96 ksi and increased to 140 ksi with the first level of confinement. The third beam, with the second level of confinement, failed due to crushing of the concrete in the compression zone prior to reaching the ultimate capacity of the splice.
## Table 4-3: Measured stresses in the spliced bars

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Cover</th>
<th>$f'_c$</th>
<th>Splice Length</th>
<th>Stirrups Spacing</th>
<th>Measured Strain Gages Readings</th>
<th>Cracked Section Analysis</th>
<th>Moment Curvature Analysis</th>
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<td></td>
<td></td>
<td>in.</td>
<td>psi</td>
<td>in.</td>
<td>kips</td>
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<td></td>
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<td>109.5</td>
<td>138.1</td>
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<td>74.4</td>
<td>107</td>
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<td></td>
<td></td>
<td>3.5</td>
<td>112.9</td>
<td>131</td>
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</tbody>
</table>
4.2.3 Load-Deflection Behavior

Typical Load-Deflection behavior reflecting the confinement effect is given in Figure 4-8 for Group 1. Test results indicate that the cracking and post cracking stiffnesses were identical for each group of beams regardless of the confinement level. The beams without confining transverse reinforcement failed due to splitting and the loss of the concrete cover over the entire length of the splice shortly after the initiation of the splitting cracks. When confining transverse reinforcement was added within the splice zone, the beams were capable of carrying more load and deflection and the failure was more ductile as shown in Figure 4-8 for beam 8-5-O-C1. Increasing the confinement, as shown for beam 8-5-O-C2, allowed the beams to achieve their ultimate strength as shown in Figure 4-8. Typical load-deflection behavior for a group of beams reinforced with No.8 bars and a group of beams reinforced with No.11 bars are shown in Figure 4-8 and Figure 4-9, respectively.
Figure 4-8: Typical Load-Deflection behavior for a group of beams reinforced with #8 bars

Figure 4-9: Typical Load-Deflection behavior for a group of beams reinforced with #11 bars
From these figures, it is obvious that increasing the confinement level increases the load-carrying capacity and the ductility of the beams. For example for the first group of beams, the measured stresses at failure for the first beam without confinement was 96 ksi and the corresponding deflection is 1.5 inches, while, using the first level of confinement for the second beam, the measured stresses at failure were 140 ksi accompanied with a very significant increase in the deformability of 3.8 inches deflection. The beam with the second level of confinement failed due to crushing of the concrete in the compression zone at a deformation of 7.49 inches. This behavior was similar for all tested 8 groups of beams which are listed in Appendix B part 2.
4.2.4 Crack Pattern

Crack width was measured using PI gages located at the ends of the splice and at mid-span of the beam specimens. Crack Comparators were also used to measure the crack width at different load levels. It was observed that the first flexural cracks occur at the two ends of the splice zone and near the location of the applied load for all tested beams where the maximum moment and shear are combined. Flexural cracks propagated downwards and increased in number associated with an increase in the crack width as the load was increased. The typical relationships between the flexural crack width and the stress in the reinforcing bars are shown in Figure 4-10 for beam 11-5-O-C2.

![Flexural Crack Behavior](image-url)
Further increase in the load led to the formation of the splitting cracks that are parallel to the longitudinal bars initially on the top surface of the beams as shown in Figure 4-11 followed by splitting cracks on the side of the beam close to failure. When the splitting cracks were first observed, the corresponding load was recorded and the width was measured using the crack comparators.

As mentioned before, the beams without confining transverse reinforcement failed shortly after the initiation of the splitting cracks. As for the beams with confining transverse reinforcement, the propagation of the splitting cracks was observed over the splice zone before failure occurred. This propagation was monitored as shown in Figure 4-12 and the width of the splitting cracks was measured at different load levels.
Table 4-4 shows the stresses and the forces in the longitudinal bars at the initiation of the splitting cracks. It can be seen from this table that the initiation of the splitting crack occurs at almost the same level of stress in each group. This behavior indicates that the occurrence of the splitting cracks is independent of the confinement level.
Table 4-4: Summary of splitting crack pattern

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Concrete Cover</th>
<th>f'_c</th>
<th>l_d</th>
<th>Load</th>
<th>Force in the bar* F_s = f_s * Ab</th>
<th>Stress in the bar* f_s</th>
<th>Splintering Crack Width</th>
<th>Flexural Crack Width</th>
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<tbody>
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<td>1</td>
<td>8-5-O-C0</td>
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<td>73</td>
<td>58</td>
<td>0.0394</td>
<td>0.0236</td>
<td>0.0492</td>
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<td>73</td>
<td>58</td>
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<td>0.0394</td>
<td>0.0236</td>
<td>0.0492</td>
<td></td>
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<td>0.0394</td>
<td>0.0236</td>
<td>0.0492</td>
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<td>11-8-X-C0</td>
<td>43</td>
<td>57</td>
<td>80</td>
<td>0.0059</td>
<td>0.0059</td>
<td>0.0059</td>
<td>0.0059</td>
<td>0.0295</td>
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<td></td>
<td>11-8-X-C1</td>
<td>44</td>
<td>57</td>
<td>80</td>
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<td>0.0059</td>
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</tr>
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<td></td>
<td>11-8-X-C2</td>
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<td></td>
<td></td>
<td>0.0059</td>
<td>0.0059</td>
<td>0.0059</td>
</tr>
</tbody>
</table>

* Based on Moment-Curvature Analysis.
Figure 4-13 shows that when the reinforcing bar is subjected to an axial load "P", the lugs come in contact with the concrete and exert bearing forces on the surrounding concrete to resist the applied load. The vertical component of this force causes the splitting cracks in the concrete. As the applied loads increase, the splitting forces increase and initiation of the splitting cracks occurs when the splitting forces induce tensile stress that exceeds the tensile strength of the concrete. This mechanism is also verified by the measured high stress level in the No.8 bars in comparison to No.11 bars at the initiation of the splitting. Since the splitting force is a function of the bar diameter, splitting occurs at lower stress level for large bar diameters.

![Diagram of internal forces between the reinforcing bar and the surrounding concrete](image)

Figure 4-13: Internal forces between the reinforcing bar and the surrounding concrete

Figure 4-14 shows typical behavior of splitting crack width of beams reinforced with No.11 bars. From this figure it can be seen that in general the crack width was controlled due to the presence of confining transverse reinforcement, hence achieving higher forces in the spliced bars for the same crack width compared to the beams with less or without confining transverse reinforcement. Figure 4-14 indicates that at splitting crack width of 0.02 in., the beams without confinement, with
first level of confinement and higher confinement levels achieved stresses of 54, 79, and 104 ksi, respectively. The figure also shows that the confining transverse reinforcement does not affect the initiation of the splitting cracks; however, it only restrains their propagation and width at the same load level. Splitting cracks for all tested beams in this group occurred at stress level of 32 ksi in the No.11 bars.

![Figure 4-14: Splitting crack width for the sixth group of beams](image)

Similar behavior was observed for the beams reinforced with No.8 bars where splitting crack width of 0.013 inches occurred at the corresponding stress levels for unconfined beam, first level of confinement and second level of confinement of 91, 96 and 101 ksi, respectively as shown in Figure 4-15. Initiation of the splitting cracks was observed at stress level of 75 ksi in the No.8 bars.
Behavior of the splitting cracks width for all tested beams are given in Appendix B, part 3.
4.3 Analysis

Overview

The test results of the beams were compared to evaluate the effect of the different parameters believed to affect the bond characteristics of the High Strength Steel with concrete. From the test results, it is shown that increasing the confinement level increases the load carrying capacity and the ductility of the members. The results obtained from the beams without confining transverse reinforcement will be used to evaluate the effect of the development length and the concrete cover. The resulting stresses were normalized to the quadratic root of the concrete compressive strength to eliminate the effect of $f'_c$ on the bond characteristics. The quadratic root was chosen because it has been documented by previous researchers that it gives a better representation of the effect of the concrete strength on the bond characteristics than the square root (Darwin et al., 1996; Zuo and Darwin, 2000; ACI 408R-03)

4.3.1 Effect of the Development Length

To study the effect of the development length, the beams without confining transverse reinforcement and having the same cross section were compared with each other as shown in Table 4-5. From this table, it can be seen that increasing the splice length increases the splice strength; however, the relationship is not linear. Comparing test results of groups 1 and 2 indicates that increasing the splice length from 31 in. to 41 in. which is an increase of 32%, the increase in the measured stress, $f_s$, in the bars at failure was only 16%. 
Table 4-5: Effect of the development length

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Cover</th>
<th>$f_c$</th>
<th>Splice Length</th>
<th>Measured Stresses</th>
<th>$\frac{f_t}{\sqrt[3]{f_c}}$</th>
<th>% of increase in strength</th>
<th>% of increase in $\sqrt{\frac{l_d}{d_b}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 1</td>
<td>8-5-O-C0</td>
<td>2.5</td>
<td>6015</td>
<td>31</td>
<td>96</td>
<td>10901</td>
<td>1.16</td>
<td>1.15</td>
</tr>
<tr>
<td>Group 2</td>
<td>8-5-X-C0</td>
<td>2.5</td>
<td>5817</td>
<td>41</td>
<td>110</td>
<td>12596</td>
<td>1.16</td>
<td>1.15</td>
</tr>
<tr>
<td>Group 3</td>
<td>8-8-O-C0</td>
<td>1.5</td>
<td>8400</td>
<td>40</td>
<td>91</td>
<td>9505</td>
<td>1.14</td>
<td>1.16</td>
</tr>
<tr>
<td>Group 4</td>
<td>8-8-X-C0</td>
<td>1.5</td>
<td>10200</td>
<td>54</td>
<td>109</td>
<td>10846</td>
<td>1.14</td>
<td>1.16</td>
</tr>
<tr>
<td>Group 5</td>
<td>11-5-O-C0</td>
<td>2</td>
<td>5344</td>
<td>69</td>
<td>74</td>
<td>8655</td>
<td>1.04*</td>
<td>1.15</td>
</tr>
<tr>
<td>Group 6</td>
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<td>4058</td>
<td>91</td>
<td>72</td>
<td>9021</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Group 7</td>
<td>11-8-O-C0</td>
<td>3</td>
<td>6070</td>
<td>43</td>
<td>78</td>
<td>8837</td>
<td>1.14</td>
<td>1.15</td>
</tr>
<tr>
<td>Group 8</td>
<td>11-8-X-C0</td>
<td>3</td>
<td>8383</td>
<td>57</td>
<td>96</td>
<td>10033</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* It should be noted that during the casting of the sixth group of beams, excessive bleeding of the concrete was observed which affected greatly the concrete compressive strength and the resulting stresses.

Similarly, comparing the results of groups 3 and 4 indicates that increasing the splice length by 35% causes the increase of the stresses at failure by 14% and for groups 7 and 8 increasing the splice length by 33% increases the splice strength by 14%. Note that the comparison between groups 5 and 6 is not taken into consideration due to the excessive bleeding observed during curing of the concrete which strongly affected the bond characteristics of the concrete. This nonlinear relationship is due to the nonlinearity of the stress distribution along the splice length which is more pronounced for relatively long splice lengths as shown in Figure 4-16. In depth investigation led to the conclusion that the splice strength is related to the square root of the ratio of the splice length to the diameter of the reinforcing bars. This behavior was also observed by Canbay and Frosch (2005) while analyzing the bond behavior of conventional steel. Table 4-5 indicates that the increase in the
stresses in the bars before failure is in the same order of magnitude of the increase of $\sqrt{\frac{l_d}{d_b}}$.

![Bond Stress Distribution]

Figure 4-16: Bond stress distribution along the splice length

### 4.3.2 Effect of the Concrete Cover

To investigate the effect of the concrete cover, test results were normalized to the square root of the ratio of the development length and the bar diameter to eliminate the effect of the development length as given in Table 4-6. This table demonstrates that increasing the concrete cover increases the splice strength but in a non-linear relationship. Comparing the test results of the beam in group 1 to the beam in group 3, it can be seen that increasing the concrete cover by 67% only increases the normalized splice strength by 30%. Similarly, the normalized splice strength between groups 2 and 4 indicates that increasing the splice strength is increased by 33% for an increase in the concrete cover of 67%. As for the comparison between groups 5 and 7, the increase in the concrete cover of 50% led to an increase in the normalized splice strength of 29%. Further investigation led to
the conclusion that the normalized splice strength is related to the square root of the ratio of the thickness of the concrete cover to the diameter of the reinforcing bar.

Table 4-6: Effect of the concrete cover

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Cover</th>
<th>$f_c'$</th>
<th>Splice Length</th>
<th>Measured Stresses</th>
<th>$\frac{f_s}{\sqrt[3]{f_c' \cdot \sqrt{\frac{l_d}{d_b}}}}$</th>
<th>% of increase in strength</th>
<th>% of increase in $\sqrt{\frac{c}{d_b}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 1</td>
<td>8-5-O-C0</td>
<td>2.5</td>
<td>6015</td>
<td>31</td>
<td>96</td>
<td>1958</td>
<td>1.30</td>
<td>1.29</td>
</tr>
<tr>
<td>Group 3</td>
<td>8-8-O-C0</td>
<td>1.5</td>
<td>8400</td>
<td>40</td>
<td>91</td>
<td>1503</td>
<td>1.30</td>
<td>1.29</td>
</tr>
<tr>
<td>Group 2</td>
<td>8-5-X-C0</td>
<td>2.5</td>
<td>5817</td>
<td>41</td>
<td>110</td>
<td>1967</td>
<td>1.33</td>
<td>1.29</td>
</tr>
<tr>
<td>Group 4</td>
<td>8-8-X-C0</td>
<td>1.5</td>
<td>10200</td>
<td>54</td>
<td>109</td>
<td>1476</td>
<td>1.33</td>
<td>1.29</td>
</tr>
<tr>
<td>Group 5</td>
<td>11-5-O-C0</td>
<td>2</td>
<td>5344</td>
<td>69</td>
<td>74</td>
<td>1237</td>
<td>1.29</td>
<td>1.22</td>
</tr>
<tr>
<td>Group 7</td>
<td>11-8-O-C0</td>
<td>3</td>
<td>6070</td>
<td>43</td>
<td>78</td>
<td>1600</td>
<td>1.29</td>
<td>1.22</td>
</tr>
<tr>
<td>Group 6</td>
<td>11-5-X-C0</td>
<td>2</td>
<td>4058</td>
<td>91</td>
<td>72</td>
<td>1123</td>
<td>1.41</td>
<td>1.22</td>
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<tr>
<td>Group 8</td>
<td>11-8-X-C0</td>
<td>3</td>
<td>8383</td>
<td>57</td>
<td>96</td>
<td>1578</td>
<td>1.41</td>
<td>1.22</td>
</tr>
</tbody>
</table>

Similar behavior was observed by Canbay and Frosch (2005) while investigating the effect of the concrete cover on the bond characteristics of conventional steel.

Based on these findings, the test results obtained were normalized to the square root of the ratio of the concrete cover to the bar diameter. In addition, the results provided by the University of Kansas and the University of Texas at Austin in their ongoing research on the bond characteristics of High Strength Steel were used to further investigate this behavior. These results are presented in Table 4-7 and Figure 4-17 showing a graphical scatter of the normalized stresses.
Table 4-7: Normalized stresses from the test results

<table>
<thead>
<tr>
<th>University</th>
<th>Beam ID</th>
<th>$d_b$</th>
<th>Cover</th>
<th>$f_c'$</th>
<th>Splice Length</th>
<th>Measured Stresses</th>
<th>$f_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>in.</td>
<td>in.</td>
<td>psi</td>
<td>in.</td>
<td></td>
<td>ksi</td>
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<td>NCSU</td>
<td>8-5-O-C0</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</td>
<td>1</td>
<td>2.5</td>
<td>5817</td>
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<td>110</td>
<td>1244</td>
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<tr>
<td></td>
<td>8-8-O-C0</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</td>
<td>1</td>
<td>1.5</td>
<td>10200</td>
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<td>109</td>
<td>1205</td>
</tr>
<tr>
<td></td>
<td>11-5-O-C0</td>
<td>1.41</td>
<td>2</td>
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<td>1039</td>
</tr>
<tr>
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<td>11-5-X-C0</td>
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<td>11-8-X-C0</td>
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<td>8383</td>
<td>57</td>
<td>96</td>
<td>1082</td>
</tr>
<tr>
<td>UT</td>
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<td>33</td>
<td>80</td>
<td>1184</td>
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<tr>
<td></td>
<td>UT-5-5-X-C0-3/4</td>
<td>0.625</td>
<td>0.75</td>
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<td>91</td>
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<td>0.625</td>
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<td>18</td>
<td>88</td>
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<td>25</td>
<td>110</td>
<td>1448</td>
</tr>
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<td>97</td>
<td>1274</td>
</tr>
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<td>40</td>
<td>80</td>
<td>1082</td>
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<td>72</td>
<td>1095</td>
</tr>
<tr>
<td></td>
<td>UT-8-8-X-C0-1.5</td>
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<td>1.5</td>
<td>7800</td>
<td>54</td>
<td>86</td>
<td>1017</td>
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<td></td>
<td>UT-8-5-O-C0-1.5</td>
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<td>5000</td>
<td>47</td>
<td>74</td>
<td>1048</td>
</tr>
<tr>
<td></td>
<td>UT-8-5-X-C0-1.5</td>
<td>1</td>
<td>1.5</td>
<td>4700</td>
<td>62</td>
<td>82</td>
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</tr>
<tr>
<td></td>
<td>UT-11-5-O-C0-3</td>
<td>1.41</td>
<td>2.75</td>
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<td>50</td>
<td>75</td>
<td>1072</td>
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<tr>
<td></td>
<td>UT-11-5-X-C0-3</td>
<td>1.41</td>
<td>2.75</td>
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<tr>
<td>KU</td>
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<td>47</td>
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<td>1115</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>
As shown in Figure 4-17, the average normalized stress value of the tested specimens in this program and others is 1138 with a coefficient of variation of 0.11. For simplification and to be on the conservative side, this constant will be taken as 1000. It should be noted that the results in Figure 4-17 are for splice beams using High Strength Steel. The stresses of the spliced bars measured at failure are normalized to the quadratic root of the concrete compressive strength, the square root of the ratio of development length to the bar diameter and the square root of the ratio of the concrete cover to the bar diameter, \( \frac{f_s}{\sqrt{f_c} \sqrt{\frac{l_d}{d_b}} \sqrt{\frac{c}{d_b}}} \).
This relationship can be further simplified as

$$f_s = \frac{1000}{d_b} \sqrt[3]{f'_{c}} \sqrt{l_d c}$$

Where;

- $f_s$ = Stress in the spliced bars at failure, psi;
- $f'_{c}$ = Concrete compressive strength, psi;
- $l_d$ = Splice length, in.;
- $c$ = Concrete cover, in.;
- $d_b$ = Diameter of the reinforcing bar, in.;

This proposed equation can be used to evaluate the stresses at failure in the uncoated, bottom-cast spliced reinforcing bars without confining transverse reinforcement.

Furthermore, the development lengths required to develop various stress levels in the spliced bars were calculated based on the proposed equation and the equations proposed by the ACI 318-05 and the ACI 408R-03 to assess the accuracy of the proposed equation as shown in Figure 4-18 to Figure 4-21. Similarly, the ratio of $l_d / d_b$ was evaluated for the same bar diameter to develop various stress levels using the above mentioned equations as shown in Figure 4-22 to Figure 4-24.
Figure 4-18: Required development length to develop a stress of 60 ksi for different bar sizes

Figure 4-19: Required development length to develop a stress of 70 ksi for different bar sizes
Figure 4-20: Required development length to develop a stress of 80 ksi for different bar sizes

Figure 4-21: Required development length to develop a stress of 90 ksi for different bar sizes
Bar No.5

$\frac{l_d}{d_b}$ vs Stress (ksi)

- ACI 318-05
- ACI 408R-05
- Proposed Equation

$f_c = 5000$ psi
$c = 2$ in.

Figure 4-22: Required $l_d/d_b$ to develop various stress levels for bar size No.5

Bar No.8

$\frac{l_d}{d_b}$ vs Stress (ksi)

- ACI 318-05
- ACI 408R-05
- Proposed Equation

$f_c = 5000$ psi
$c = 2$ in.

Figure 4-23: Required $l_d/d_b$ to develop various stress levels for bar size No.8
Figure 4-24: Required $\frac{l_d}{d_b}$ to develop various stress levels for bar size No.11

From these figures, it can be seen that the development lengths required by the different equations up to stress level of 90 ksi are approximately the same. For higher stress levels, both equations proposed by ACI 318 and ACI Committee 408 underestimate the development length required since they were not calibrated for high strength reinforcing bars.

Further investigations are still needed to evaluate the accuracy and the applicability of the proposed equation.
4.4 Predictions According to Codes

Overview

In this section, the measured stresses in the splices from the experimental program were compared to the predicted stresses according to different codes to assess the applicability of the code equations in the case of using High Strength Steel as longitudinal reinforcement. The equations evaluated in this section are the equations proposed by the ACI 318 (2005), the ACI Committee 408 (2003), the Eurocode 2 (2004) and the Australian Code (1998).

4.4.1 ACI 318

The development length, $l_d$, is given by ACI 318-05 in chapter 12, equation (12-1), as:

$$l_d = \left( \frac{3}{40} \frac{f_y}{f_c} \left( \frac{c_d + K_y}{d_b} \right) \right) d_b$$

Where:

- $f_y$ = The yield strength of the longitudinal reinforcement, psi;
- $f_c$ = The concrete compressive strength, psi;
- $d_b$ = The longitudinal bar diameter, in.;
- $\psi_t$ = The reinforcement location factor, taken as 1.0 for bottom cast bars;
- $\psi_c$ = The coating factor, taken as 1.0 for uncoated reinforcement;
\[ \psi_s = \text{The bar size factor, taken as 1.0 for No.7 bars and larger;} \]

\[ \lambda = \text{A factor reflecting the lower tensile strength of lightweight concrete and the resulting reduction of the splitting resistance, which increases the development length in lightweight concrete, taken as 1.0 for normal weight concrete;} \]

\[ k_{tr} = \text{The factor that represents the contribution of the confining transverse reinforcement across potential splitting cracks;} \]

\[ k_{tr} = \frac{A_{tr} f_{yt}}{1500 s n} \]

Where;

\[ A_{tr} = \text{The area of transverse reinforcement index perpendicular to the plane of splitting;} \]

\[ f_{yt} = \text{The yield strength of the transverse reinforcement;} \]

\[ s = \text{The spacing of the transverse reinforcement in the splice zone;} \]

\[ n = \text{The number of bars being spliced or developed along the plane of splitting;} \]

\[ c_o = \text{The factor that represents the smallest of the side cover, the cover over the bar (in both cases measured to the center of the bar) or one half of the center to center spacing of the bars;} \]

It should be noted that the term \[ \frac{c_o + k_{tr}}{d_b} \] is limited to a value of 2.5 to ensure splitting failure rather than pullout failure. Table 4-8 Table 4-1 summarizes the predicted stresses determined by ACI 318-05 equations.
### Table 4-8: Stresses predicted by the ACI 318 equations

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>d_b</th>
<th>c</th>
<th>$f_e'$</th>
<th>$\psi_t$</th>
<th>$\psi_c$</th>
<th>$\psi_s$</th>
<th>$\lambda$</th>
<th>S</th>
<th>ktr</th>
<th>$\frac{c_b + k_{tr}}{d_b}$</th>
<th>$l_d$</th>
<th>f_s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 1</td>
<td>8-5-O-C0</td>
<td>0.0</td>
<td>0.0</td>
<td>3.0</td>
<td>2.5</td>
<td>80</td>
<td>31</td>
<td>80</td>
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<td></td>
<td></td>
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</tr>
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4.4.2 ACI Committee 408

The development length, \( l_d \), to bar diameter, \( d_b \), ratio can be determined using the following equation proposed by the ACI Committee draft report 408R-03 as:

\[
\frac{l_d}{d_b} = \frac{\left( \frac{f_y}{f_c^{1/4}} - 2400\omega \right) \alpha \beta \lambda}{76.3 \left( \frac{c \omega + K_{tr}}{d_b} \right)}
\]

Where;

\( f_y = \) The yield strength of the reinforcing steel which could be replaced by \( f_x \) to evaluate the stress in the reinforcing bar for a given splice length \( l_d \), psi;

\( f_c' = \) The concrete compressive strength, psi;

\( \alpha = \) The reinforcement location factor taken as unity as all the bars were bottom cast;

\( \beta = \) The coating factor, taken as 1.0 for uncoated bars;

\( \lambda = \) The lightweight concrete factor taken as unity;

\( c = \) The concrete cover, in.;

\( \omega = \) A factor that depends on the minimum and maximum concrete cover;

\[
\omega = 0.1 \frac{c_{\text{max}}}{c_{\text{min}}} + 0.9 \leq 1.25
\]

\( k_{tr} = \) Transverse reinforcement index;
\[ K_w = \left(\frac{0.52t_tr_dA_w}{sn}f_c^{1/2}\right) \]

\[ t_r = \text{Term representing the effect of the relative rib area, } R_r, \text{ on the contribution of the transverse reinforcement on the bond strength;} \]

\[ t_r = 9.6R_r + 0.28 \leq 1.72 \]

\[ t_d = \text{Term representing the effect of the bar diameter on the contribution of the transverse reinforcement on the bond strength;} \]

\[ t_d = 0.78d_b + 0.22 \]

\[ s = \text{The spacing of the transverse reinforcement in the splice region, in.;} \]

\[ n = \text{The number of bars being developed or spliced;} \]

It should be mentioned that the value of \( \frac{c_\omega + K_w}{d_b} \) should be less than 4 for conventional reinforcement with yield strength limited to 60 ksi to assure splitting failure rather than pullout failure according to ACI 408R-03. Solving the above equation for \( f_s \), the stress in the reinforcing bar can be predicted using the equation proposed by ACI Committee 408. Table 4-9 summarizes the stress values determined by the ACI 408R-03 equations for the tested beams.
### Table 4-9: Stresses predicted by the ACI Committee 408 equations

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<th>$l_d$</th>
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4.4.3 Eurocode 2

The equation used to evaluate the stresses in the reinforcing bars according to the Eurocode Standard EN 1992-1-1:2004: Design of concrete structures, Part 1-1; General rules and rules for building is as follows:

\[ l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 l_{b,\text{rd}} \geq l_{b,\text{min}} \]

Where;

\( l_{bd} \) = The design anchorage length, mm;

\( \alpha_1 \) = Effect of the form of the bars assuming adequate cover, taken as 1.0 for straight bars;

\( \alpha_2 \) = Effect of the concrete minimum cover, given by:

\[ \alpha_2 = 1 - 0.15(c_d - \phi)/\phi \text{ and } 0.7 \leq \alpha_2 \leq 1.0; \]

\( \alpha_3 \) = Effect of confinement by transverse reinforcement, given by:

\[ \alpha_3 = 1 - K \left( \sum A_{st} - \sum A_{st,\text{min}} \right) / A_s \]

Where;

\( K \) = 0.1 for the present case;

\( \sum A_{st} \) = The cross-sectional-area of the transverse reinforcement along the design anchorage length \( l_{bd} \);

\( \sum A_{st,\text{min}} \) = The cross-sectional area of the minimum transverse reinforcement which is \( 0.25A_s \) for beams;

\( \alpha_4 \) = The influence of one or more welded transverse bars along the design anchorage length \( l_{bd} \) and is assumed to be 1.0 for this study;
\( \alpha_s \) = The effect of the pressure transverse to the plane for splitting along the anchorage length and is assumed to be 1.0 for this study;

\( l_{b,qd} \) = The basic required anchorage length given by;

\[
l_{b,qd} = \frac{\phi \sigma_{sd}}{4 f_{bd}}
\]

Where;

\( \phi \) = The longitudinal bar diameter, mm;

\( \sigma_{sd} \) = The design stress of the bar at the position starting where the anchorage is measured, MPa;

\( f_{bd} \) = The design value of the ultimate bond stress for concrete taken from the equation:

\[
f_{bd} = 2.25 \eta f_{cd}
\]

Where;

\( \eta_1 \) = A coefficient related to the quality of the bond condition and the position of the bar during concreting, taken as 1.0 for good conditions;

\( \eta_2 \) = A term related to the bar diameter where;

\[
\eta_2 = \begin{cases} 
1.0 & \text{for } \phi \leq 32 \text{mm} \\
(132 - \phi) / 100 & \text{for } \phi > 32 \text{mm}
\end{cases}
\]

\( f_{cd} \) = The design value of concrete tensile strength given by the equation:

\[
f_{cd} = \alpha_c f_{cd,0.05} / \gamma_c
\]

Where;

\( \alpha_c \) = A coefficient taking account of long term effects on the tensile strength
and of unfavorable effects, resulting from the applied load configuration
and is assumed to be 1.0 as recommended by the code provisions;

\( \gamma_c \) = The partial safety factor for concrete and assumed to be 1.5 for this study;

\( f_{ck,0.05} \) = The concrete tensile strength with 5\% fractile, MPa, where;

\[ f_{ck,0.05} = 0.7f_{ctm} \]

Where;

\( f_{ctm} \) = The mean value of axial tensile strength of concrete, MPa, where:

\[ f_{ctm} = 0.30f_{ck}^{(2/3)} \leq C50/60 \text{ where } C \text{ is the concrete grade,} \]

\[ f_{ctm} = 2.12 \ln \left(1 + \left(\frac{f_{cm}}{10}\right)\right) > C50/60 \]

\( f_{ck} \) = The characteristic compressive cylinder strength of concrete at 28 days, MPa;

\( f_{cm} \) = The mean value of concrete cylinder compressive strength, MPa;

It should be noted that the Eurocode requires a minimum amount of
transverse reinforcement in the splice zone, which is not the case for the ACI
requirements. The Eurocode states: “where the diameter, \( \phi \), of the lapped bars is
greater than or equal to 20 mm. (0.78 in.), the transverse reinforcement should have
a total area \( A_{st} \) (sum of all legs parallel to the layer of the spliced reinforcement) not
less than the area \( A_s \) of one lapped bar (\( \Sigma A_{st} \geq 1.0A_s \)).” Table 4-10 summarizes the
predicted stress values in the reinforcing bars for the beams tested.
Table 4-10: Stresses predicted by the Eurocode 2 equations

| Group | Beam ID       | Bar | C   | \( l_{bd} \) | \# of str | \( A_{st} \) | \( A_s \) | \( \alpha_1 \) | \( \alpha_2 \) | \( \alpha_3 \) | \( l_{brqd} \) | \( f_{ck} \) | \( f_{bd} \) | Stresses |
|-------|---------------|-----|-----|-------------|------------|-------------|---------|-------------|-------------|-------------|-------------|---------|---------|----------|----------|
|       |               | \( \phi \) | mm. | mm. | mm. | mm. | mm. | mm. | MPa | MPa | MPa | ksi |        |          |          |
| Group 1 | 8-5-O-C0     | 63.5 | 787 | 0  | 0.0 | 10 | 1266.8 | 506.7 | 1   | 0.775 | 1.000 | 1016 | 41   | 3.77 | 604 | 88      |
|        | 8-5-O-C1     |      |     |    |     |    |       |       |     |       |       |     |      |      |      |        |
|        | 8-5-O-C2     |      |     |    |     |    |       |       |     |       |       |     |      |      |      |        |
| Group 2 | 8-5-X-C0     | 25.4 | 1041| 0  | 0.0 | 11 | 1393.4 | 506.7 | 1   | 0.750 | 1.000 | 1344 | 41   | 3.77 | 1041 | 151     |
|        | 8-5-X-C1     |      |     |    |     |    |       |       |     |       |       |     |      |      |      |        |
|        | 8-5-X-C2     |      |     |    |     |    |       |       |     |       |       |     |      |      |      |        |
| Group 3 | 8-8-O-C0     | 38.1 | 1016| 0  | 0.0 | 13 | 1646.8 | 506.7 | 1   | 0.700 | 1.000 | 1098 | 58   | 4.51 | 780 | 113     |
|        | 8-8-O-C1     |      |     |    |     |    |       |       |     |       |       |     |      |      |      |        |
| Group 4 | 8-8-X-C0     | 1372| 1372| 0  | 0.0 | 21 | 2660.2 | 506.7 | 1   | 0.500 | 1.000 | 1483 | 70   | 4.85 | 1132 | 164     |
|        | 8-8-X-C1     |      |     |    |     |    |       |       |     |       |       |     |      |      |      |        |
| Group 5 | 11-5-O-C0    | 50.8 | 1753| 0  | 0.0 | 13 | 1646.8 | 506.7 | 1   | 0.700 | 1.000 | 1881 | 37   | 3.39 | 729 | 106     |
|        | 11-5-O-C1    |      |     |    |     |    |       |       |     |       |       |     |      |      |      |        |
|        | 11-5-O-C2    |      |     |    |     |    |       |       |     |       |       |     |      |      |      |        |
| Group 6 | 11-5-X-C0    | 34.9 | 2311| 0  | 0.0 | 12 | 1520.1 | 506.7 | 1   | 0.721 | 1.000 | 2481 | 28   | 2.82 | 801 | 116     |
|        | 11-5-X-C1    |      |     |    |     |    |       |       |     |       |       |     |      |      |      |        |
|        | 11-5-X-C2    |      |     |    |     |    |       |       |     |       |       |     |      |      |      |        |
| Group 7 | 11-8-O-C0    | 958.0 | 1092| 0  | 0.0 | 10 | 1266.8 | 506.7 | 1   | 0.893 | 1.000 | 1328 | 42   | 3.69 | 560 | 81      |
|        | 11-8-O-C1    |      |     |    |     |    |       |       |     |       |       |     |      |      |      |        |
|        | 11-8-O-C2    |      |     |    |     |    |       |       |     |       |       |     |      |      |      |        |
| Group 8 | 11-8-X-C0    | 1448| 1448| 0  | 0.0 | 19 | 2406.9 | 506.7 | 1   | 0.774 | 1.000 | 1760 | 58   | 4.38 | 1140 | 165     |
|        | 11-8-X-C1    |      |     |    |     |    |       |       |     |       |       |     |      |      |      |        |
|        | 11-8-X-C2    |      |     |    |     |    |       |       |     |       |       |     |      |      |      |        |
4.4.4 The Australian Code

The development length for a deformed bar, \( L_{sy,t} \), according to the Australian code can be determined as follows:

\[
L_{sy,t} = k_1 k_2 \frac{A_b f_{yw}}{C \sqrt{f_c}} \geq 25 k_1 d_b
\]

Where;

\( k_1 = \) A factor that accounts for the bar location with a value of 1.0 for bottom cast bars;

\( k_2 = \) A factor that depends on the type of member, the bar and the mode of failure. The value of this factor also depends on the presence of any transverse reinforcement or not. The code recommends a value of \( k_2 \) of 2.2 when the transverse reinforcement is used and 2.4 in case there is no transverse reinforcement;

\( A_b = \) The cross-sectional area of the anchored bars, mm\(^2\);

\( C = \) The diameter of the concrete annulus, which is a fictional area surrounding the bar where:

\[
C = 2c + d_b \text{ where;}
\]

\( c = \) The concrete cover, mm;

\( f_{yw} = \) The yield strength of the reinforcing steel, MPa;

\( f_c' = \) The concrete compressive strength, MPa;

A minimum value of \( 25k_1 d_b \) is specified to ensure that the bar does not prematurely pullout prior to the mobilization of the concrete around the bar in...
providing an effective means of stress development. When the equation is solved for the steel stress, the following predicted values are given in Table 4-11.
## Table 4-11: Stresses predicted by the Australian Code

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Bar c</th>
<th>$f'_c$</th>
<th>$l_{bd}$</th>
<th>S</th>
<th>$k_1$</th>
<th>$k_2$</th>
<th>C</th>
<th>$A_b$</th>
<th>Stresses</th>
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<td></td>
<td></td>
<td>φ</td>
<td>mm.</td>
<td>MPa</td>
<td>mm.</td>
<td></td>
<td></td>
<td>mm²</td>
<td>MPa</td>
<td>ksi</td>
</tr>
<tr>
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<td>152.4</td>
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<td>597.78</td>
<td>979</td>
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4.4.5 Analysis of the Code Prediction

The stresses in the reinforcing bars obtained from the cracked section analysis are compared with those predicted by the equations proposed by the ACI 318, the ACI 408R, the Eurocode and the Australian code as shown in Table 4-12. This table also provides the ratios of the calculated stresses from the cracked section analyses to the predicted stresses according to the code equations. The scatter of the ratio of the calculated / predicted stresses for each of the code equations evaluated is shown in Figure 4-25 to Figure 4-28, followed by Figure 4-29 showing the distribution of test / prediction ratio for ACI 318, ACI 408R, Eurocode and the Australian Code side by side to compare their accuracy.

![Figure 4-25: Scatter of the Measured / Predicted stress ratio for the ACI 318 equation](image)

- **ACI 318**
  - Average = 1.18
  - Standard Deviation = 0.36
  - Coefficient of variation = 0.30
  - Maximum Ratio = 1.92
  - Minimum Ratio = 0.69

Figure 4-25: Scatter of the Measured / Predicted stress ratio for the ACI 318 equation
Table 4-12: Comparison of the test results to the code equations

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Calculated Stresses</th>
<th>ACI 318</th>
<th>ACI 408</th>
<th>Eurocode</th>
<th>Australian Code</th>
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<td></td>
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<td>Stress</td>
<td>Ratio</td>
<td>Stress</td>
<td>Ratio</td>
<td>Stress</td>
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<td>80</td>
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<td>84</td>
<td>1.15</td>
<td>88</td>
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<tr>
<td>8-5-O-C1</td>
<td>140</td>
<td>80</td>
<td>1.75</td>
<td>104</td>
<td>1.34</td>
<td>113</td>
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<td>8-5-O-C2</td>
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<td>80</td>
<td>1.90</td>
<td>104</td>
<td>1.46</td>
<td>167</td>
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<td>104</td>
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<td>103</td>
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<td>0.76</td>
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<td>123</td>
<td>1.04</td>
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</table>
Figure 4-26: Scatter of the Measured / Predicted stress ratio for the ACI 408R equation

Average = 1.09
Standard Deviation = 0.17
Coefficient of variation = 0.16
Maximum Ratio = 1.48
Minimum Ratio = 0.76

Figure 4-27: Scatter of the Measured / Predicted stress ratio for the Eurocode equation

Average = 0.93
Standard Deviation = 0.21
Coefficient of variation = 0.22
Maximum Ratio = 1.42
Minimum Ratio = 0.62
Figure 4-28: Scatter of the Measured / Predicted stress ratio for the Australian Code equation

Figure 4-29: Distribution of the measured / predicted stress ratios for the 4 code equations
From the results presented in Table 4-12, it can be seen that the ACI 408R equation gives the closest predictions to the resulting stresses among all equations provided by the other codes. The average of the ratios of the calculated / predicted stresses from the ACI 408R equation is 1.09 compared to 1.18 for the ACI 318, 0.93 for the Eurocode and 1.11 for the Australian code. In addition the ACI 408R equation gives the closest coefficient of variation, 0.16, compared to 0.30, 0.22 and 0.25 for the ACI 318, the Eurocode and the Australian code, respectively.

The comparison between the calculated stresses and the predicted stresses by the code equations suggest that the equations proposed by ACI 318 and ACI 408R underestimate the effect of confinement especially when High Strength Steel is used. Similar observations were made by Canbay and Frosch (2005) when they calculated the average steel stress in the stirrups to be approximately 9000 psi for beams reinforced with conventional steel. This behavior can be attributed to the high stresses in the reinforcing bars. These high stresses induce high bearing forces on the concrete in the vicinity of the ribs which can be translated into 2 components: the horizontal component, $F_{\text{longitudinal}}$, which causes the bond forces with the concrete, and the vertical component, $F_{\text{splitting}}$, which causes the splitting forces. These splitting forces cause high mobilization in the confining transverse reinforcement resulting in higher stresses in the stirrups.

It can also be concluded from the comparison of the measured / predicted ratios of the ACI 318 and the ACI 408R equations that the ACI 318 is more conservative than the ACI 408R equation. This is due to the fact that the ACI 408R equation uses the quadratic root of the concrete compressive strength to evaluate
the effect of $f'_c$ on the development length, compared to the square root of $f'_c$ used in the equation proposed by ACI 318. The quadratic root has been proven to be a better representative of the effect of the concrete compressive strength on the bond characteristics than the square root as documented by many researchers including Darwin et al. (1996) and Zuo and Darwin (2000).

Test results suggest that, given adequate concrete cover and development length, a stress value of 90 ksi can be achieved in the MMFX bars without confining transverse reinforcement for the No.8 bars and higher stresses, up to 140 ksi, can be achieved by adding confining transverse reinforcement. For the No.11 bars, the stress value that can be achieved without confining transverse reinforcement is only 70 ksi and stresses up to 140 ksi can be achieved by adding confining transverse reinforcement.

From the failure of the tested beams, it was observed that splitting failure was the governing mode of failure although the value of the term $\frac{c \omega + K_w}{d_h}$ from the ACI 408R-03 equation was higher than 4.0, the limit set by the code committee to ensure splitting failure as shown in Figure 4-30.
This behavior is due to the high strength of the reinforcing bars that leads to higher bearing forces exerted by the ribs on the concrete interface. These inclined forces lead to high longitudinal stresses in the reinforcing bars and to high splitting force in the concrete. Accordingly, the measured value of \( \frac{c_\omega + k_r}{d_b} \) was higher than 4.0 for beams that failed in splitting. When the limit of \( \frac{c_\omega + K_r}{d_b} \) is increased to 5.0 rather than 4.0, the results of the predicted stresses were closer to the measured ones as shown in Table 4-13. This table compares the results using the limit of 4.0 and 5.0 for the \( \frac{c_\omega + K_r}{d_b} \) parameter.
Table 4-13: Predicted stresses with different limits for $\frac{c \omega + K_r}{d_b}$

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<td>8-5-O-C1</td>
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<tr>
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<td>8-5-O-C2</td>
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<td>104</td>
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<td>8-5-X-C0</td>
<td>110</td>
<td>103</td>
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<td>8-5-X-C1</td>
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<td>8-5-X-C2</td>
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<td>Group 3</td>
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<td>155</td>
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<td>Group 5</td>
<td>11-5-O-C0</td>
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<td>82</td>
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<td>11-5-O-C2</td>
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<td>Group 6</td>
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<td>11-5-X-C1</td>
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<td>101</td>
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<td>11-8-X-C1</td>
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<td></td>
<td>11-8-X-C2</td>
<td>157</td>
<td>141</td>
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Table 4-14: Improved results with the increased limit

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<th>limit = 5.0</th>
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<td><strong>Average</strong></td>
<td>1.09</td>
<td>1.02</td>
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<tr>
<td><strong>Standard Deviation</strong></td>
<td>0.17</td>
<td>0.12</td>
</tr>
<tr>
<td><strong>Coefficient of Variation</strong></td>
<td>0.16</td>
<td>0.12</td>
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</table>

Table 4-14 shows the improved average from 1.09 to 1.02 when the limit of \( \frac{c\omega + K}{d_b} \) is increased from the 4.0 to 5.0, the improved standard deviation from 0.17 to 0.12 and the coefficient of variation from 0.16 to 0.12.

This behavior suggests that increasing the limit of \( \frac{c\omega + K}{d_b} \) from 4.0 to 5.0 will improve the predicted stresses by the equation proposed by the ACI Committee 408 in the case of High Strength Steel.
5.1. Overview

Bond between the concrete and the reinforcing steel plays a major role in the performance of reinforced concrete structures. The research undertaken investigates the bond characteristics of new commercially available high performance steel, known as MMFX Steel. The new High Performance Steel is known to have an enhanced corrosion resistance and a significantly higher strength when compared to the conventional Grade 60 steel. This thesis summarizes the research findings of an experimental program completed to investigate the bond behavior of the MMFX Steel and the concrete. The experimental program included twenty-two large scale reinforced concrete splice beams fabricated and tested at North Carolina State University. The experimental results were analyzed to describe the bond behavior of this type of steel reinforcement with the concrete and evaluate the effect of selected parameters on the bond strength of lap spliced bars. The resulting stresses were also compared with those predicted by the equations proposed by different building codes. Results from this experimental program show that bond can be achieved without confinement and high levels of stresses, including the strength of the bars can be achieved by adequate confinement. Use of High Strength reinforcement can lead to significant reduction of the material requirements for a particular project.
5.2. Conclusions

The bond behavior of High Performance reinforcing bars to concrete was found to be similar to conventional steel reinforcement in many of the following aspects:

1. The use of the quadratic root of the concrete compressive strength \( \sqrt{f_c} \) provides a better estimation of the behavior of the lapped splices as compared with the square root.

2. The increase in the splice strength is related to the increase in the development length in a non linear relationship. The strength is proportional to the square root of the ratio of the development length to the bar diameter \( \sqrt{l_d/d_b} \).

3. The relationship between the increase in the concrete cover and the increase in the splice strength is not linear. The strength is proportional to the square root of the ratio of the concrete cover to the bar diameter \( \sqrt{c/d_b} \).

The research findings of the experimental program conducted specifically to identify the bond behavior of High Performance reinforcing bars concluded the following unique results:

1. A minimum amount of transverse reinforcement should be used for members reinforced with High Strength Steel to ensure ductile behavior of the members.
2. The following simple equation is proposed to be used for the evaluation of the stresses in the spliced bars without confining transverse reinforcement:

\[ f_s = \frac{1000}{d_b} \sqrt{f'_c} \sqrt{f_c} \]

3. The equations proposed by the ACI 318 and the ACI Committee 408 underestimate the effect of confinement.

4. The equation proposed by the ACI Committee 408 predicts the closest results to those obtained in the experimental research.

5. The limit of the term \( \frac{c \omega + K_d}{d_b} \) of the equation proposed by the ACI Committee 408 to ensure splitting failure can be increased from 4.0 to 5.0 in the case of using high strength steel to provide closer predictions to the test results.

6. Members reinforced with High Strength Steel without confining transverse reinforcement exhibit brittle failure. Using transverse reinforcement, in the form of stirrups, increases the flexural carrying capacity and ductility of flexural members.

7. Given adequate concrete cover and development length, a stress value of 90 ksi can be achieved in the MMFX bars without confining transverse reinforcement for the No.8 bars and higher stresses, up to 140 ksi, can be achieved by adding confining transverse reinforcement.

8. For the No.11 bars, the stress value that can be achieved without confining transverse reinforcement is 70 ksi and stresses up to 140 ksi can be achieved by adding confining transverse reinforcement.
5.3 Recommendation for Future Research

Based on the results obtained in this experimental program, recommendations can be given to guide the future work to develop in-depth-understanding of bond behavior of high performance steel and more economical design:

1. Due to the non-linear distribution of the bond stresses along the splice length, it is anticipated that, after a certain development length, longer splices will not enhance the developed stresses and it would be more economical to add confining transverse reinforcement in order to increase the splice strength. The maximum economical development length should be determined.

2. To ensure ductile behavior of concrete members reinforced with High Performance steel, a minimum amount of confining transverse reinforcement should be determined.
ACI Committee 318, 2005, “Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (318R-05)”, American Concrete Institute, Farmington Hills, Mich., 430pp.


ACI Committee 439, 1973, “Uses and Limitations of High Strength Steel Reinforcement $f_y \geq 60$ ksi (42.2 Kgf/mm$^2$)”, American Concrete Institute, Farmington Hills, Mich., 49pp.


Azizinamini, Atorod; Pavel, Rob; Hatfield, Erleen and Ghosh, S.K., 1999, “Behavior of Lap-Spliced Reinforcing Bars Embedded in High-Strength Concrete”, ACI Structural Journal, V. 96, No.5, September-October, pp.826-835


Canbay, Erdem and Frosch, Robert J., 2005, “Bond Strength of Lap-Spliced Bars”, ACI Structural Journal, V.102, No.4, July-August, pp. 605-614.


References


Guralnick, Sidney A., 1960, “High-Strength Deformed Steel Bars for Concrete Reinforcement”, Journal of the American Concrete Institute, Title No. 57-12, September, pp. 241-282.


Sinha, Nripendra C. and Ferguson, Phil, M., 1964, “Ultimate Strength with High Strength Reinforcing Steel with an Indefinite Yield Point”, Journal of the American Concrete Institute, Title No.61-26, April, pp. 399-418.


Appendices
Appendix A

Details of the tested Beams

The detailed cross sections of the twenty two beams are given in this appendix.

Figure A-1: Reinforcement details for beam 8-5-O-C0
Figure A-2: Reinforcement details for beams 8-5-O-C1 and 8-5-O-C2
Figure A-3: Reinforcement details for beam 8-5-X-C0
Appendix A

Figure A-4: Reinforcement details for beams 8-5-X-C1 and 8-5-X-C2
Figure A-5: Reinforcement details for beam 8-8-O-C0
Figure A-6: Reinforcement details for beam 8-8-O-C1
Figure A-7: Reinforcement details for beam 8-8-X-C0
Figure A-8: Reinforcement details for beam 8-8-X-C1
Figure A-9: Reinforcement details for beam 11-5-O-C0
Figure A-10: Reinforcement details for beams 11-5-O-C1 and 11-5-O-C2
Figure A-11: Reinforcement details for beam 11-5-X-C0
Figure A-12: Reinforcement details for beams 11-5-X-C1 and 11-5-X-C2
Figure A-13: Reinforcement details for beam 11-8-O-C0
Figure A-14: Reinforcement details for beams 11-8-O-C1 and 11-8-O-C2
Figure A-15: Reinforcement details for beam 11-8-X-C0
Figure A-16: Reinforcement details for beams 11-8-X-C1 and 11-5-X-C2
Appendix B

Test Results

Part 1: Mode of Failure

Figure B1-1: Splitting Failure of Beam 8-5-O-C0

Splice Length = 31 in.
Appendix B

Figure B1-2: Splitting Failure of Beam 8-5-O-C1

Figure B1-3: Flexural Failure of Beam 8-5-O-C2
Appendix B

Figure B1-4: Splitting Crack of Beam 8-5-X-C0

Figure B1-5: Flexural Failure of Beam 8-5-X-C1
Appendix B

Figure B1-6: Flexural Failure of Beam 8-5-X-C2

Figure B1-7: Splitting Failure of Beam 8-8-O-C0
Appendix B

Figure B1-8: Splitting Failure of Beam 8-8-O-C1

Figure B1-9: Splitting Failure of Beam 8-8-X-C0
Appendix B

Figure B1-10: Splitting Failure of Beam 8-8-X-C1

Figure B1-11: Splitting Failure of Bema 11-5-O-C0
Figure B1-12: Splitting Failure of Beam 11-5-O-C1

Figure B1-13: Splitting Failure of Beam 11-5-O-C2
Appendix B

Figure B1-14: Splitting Failure of Beam 11-5-X-C0

Figure B1-15: Splitting Failure of Beam 11-5-X-C1
Appendix B

Figure B1-16: Splitting Failure of Beam 11-5-X-C2

Figure B1-17: Splitting Failure of Beam 11-8-O-C0
Appendix B

Figure B1-18: Splitting Failure of Beam 11-8-O-C1

Figure B1-19: Flexural Failure of Beam 11-8-O-C2
Figure B1-20: Splitting Failure of Beam 11-8-X-C0

Figure B1-21: Splitting Failure of Beam 11-8-X-C1
Figure B1-22: Flexural Failure of Beam 11-8-X-C2
Part 2: Load-Deflection Behavior

Figure B2-1: Load-Deflection Behavior of the First Group of beams

Figure B2-2: Load-Deflection Behavior of the Second Group of beams
Appendix B

Figure B2-3: Load-Deflection Behavior of the Third Group of beams

Figure B2-4: Load-Deflection Behavior of the Fourth Group of beams
Figure B2-5: Load-Deflection Behavior of the Fifth Group of beams

Figure B2-6: Load-Deflection Behavior of the Sixth Group of beams
Figure B2-7: Load-Deflection Behavior of the Seventh Group of beams

Figure B2-8: Load-Deflection Behavior of the Eight Group of beams
Appendix B

Part 3: Splitting Crack Pattern

Figure B3-1: Splitting Crack Pattern of the First group of beams

Figure B3-2: Splitting Crack Pattern of the Second group of beams
Figure B3-3: Splitting Crack Pattern of the Third group of beams

Figure B3-4: Splitting Crack Pattern of the Fourth group of beams
Figure B3-5: Splitting Crack Pattern of the Fifth group of beams

Figure B3-6: Splitting Crack Pattern of the Sixth group of beams
Appendix B

Figure B3-7: Splitting Crack Pattern of the Seventh group of beams

Figure B3-8: Splitting Crack Pattern of the Eighth group of beams