ABSTRACT

ELAGROUDY, HOSSAM ALY. BOND CHARACTERISTICS OF MICRO-COMPOSITE MULTI-STRUCTURAL FORMABLE STEEL USED IN REINFORCED CONCRETE STRUCTURES (under the direction of Dr. Sami Rizkalla.)

The bond performance of a unique type of reinforcing steel rebars, claimed to have high corrosion resistance as well as high tensile strength, with concrete was studied. The objective was to investigate the bond behavior of straight rebars made out of this steel, named MMFX, embedded in concrete flexural members and to examine the applicability of the current expressions for bond force to predict the bond capacity of the MMFX bars embedded in concrete. Two phases of experimental investigation was conducted. In the first phase, four beam end specimens were tested and in the second phase eight splice beams were studied. The bond behavior of the MMFX steel bars was found to be similar to that of carbon steel. The bond strength of the MMFX is significantly reduced as the tensile stresses developed in the bar went beyond the proportional limit. Both the ACI code 318-02 equation for bond force and the current equation proposed by the ACI committee 408 for bond force gave conservative prediction for bond force for low stress levels. However, at high stress levels, the prediction of the two equations went to the unconservative side. The non-linear behavior of the MMFX stress-strain curve was the reason behind the unconservative prediction. The above two equations were modified to ensure conservative prediction at high stress levels. A second degree best fitting curve was found to be the best to describe the relationship between the splice length and the bond force capacity for both #6 and #8 MMFX bars.
BOND CHARACTERISTICS OF MICRO-COMPOSITE MULTI-STRUCTURAL FORMABLE STEEL USED IN REINFORCED CONCRETE STRUCTURES

By

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APPROVED BY:

____________________________
Chair of Advisory Committee

July 24, 2003
DEDICATION

It is my pleasure to dedicate this Thesis to my beloved parents for all the overwhelming inspiration and the overseas support they generously showed to me throughout the two full years of work towards my Master of Science degree. I do not know how to thank them but I will ask Allah to bless them, guard them, and give them long and happy life here and in the hereafter.

" وأن أشْكُرُ لِي وَلَوْ الْدِّيْكَ إِلَيْهِ التَّمِيزُ "

" وَقُلْ رَبِّ ارْحَمْهُمَا كَمَا رَبِّي صَغِيرًا "
PERSONAL BIOGRAPHY

Hossam Elagroudy was born in Cairo, Egypt, in October 1976. Throughout the school years he was always among the top students. He joined Ain Shams University, the number one University in Egypt, in Engineering to study towards a Bachelor degree in civil engineering the year. He graduated in July 1999 with a GPA 3.9 and a score 93% and a number one rank over more than 250 colleagues.

After graduation and as a result of his outstanding performance, he was elected as an assistant teacher in Ain Shams University that represents the beginning of the road towards a faculty member in this University. In the year 2001 and in order to enhance and widen his knowledge in the field of Civil Engineering, he decided to join North Carolina State University which is one of the best Universities in the field of Civil Engineering in the United States. He started his graduate program towards the Master of Science degree in the fall of the same year by working as a research assistant in the Constructed Facilities Laboratory under the supervision of Dr. Sami Rizkalla. In fall 2003, he was awarded his Master of Science degree. He is planning to continue his academics studies towards a Ph.D. degree in the same field from the University of Toronto in Canada beginning in the fall of 2003.
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LIST OF SYMBOLS

\( A_b: \)  bar cross sectional area (in.\(^2\))

\( A_s: \)  total area of steel (in.\(^2\))

\( A_{tr}: \)  area stirrup or tie crossing the potential plane of splitting. (in.\(^2\))

\( b: \)  width of the beam web (in.)

\( C'_{\text{min}}: \) minimum of concrete covers (bottom and side) surrounding the bar and half the clear spacing between bars, minimum of \((C_b \text{ or } C_{so})\) and \(C_{si}\), (in).

\( C'_{\text{max}}: \) maximum of concrete covers (bottom and side) surrounding the bar and half the clear spacing between bars, minimum of \((C_b \text{ or } C_{so})\) and \(C_{si}\), (in).

\( C_{\text{min}}: \) minimum value of \(C_s\) or \(C_b\) (in.)

\( C_{\text{max}}: \) maximum value of \(C_s\) or \(C_b\) (in.)

\( C: \)  \(C_{\text{min}} + 0.5 \, d_b\) (in.)

\( C_b: \)  thickness of the clear bottom concrete cover (in.)

\( C_{so}: \)  thickness of the clear side concrete cover (in.)

\( C_s: \)  minimum of \((C_{so}, \, C_{si} + 0.25\, \text{in})\)

\( C_{si}: \)  half the clear spacing between spliced/developed bars (in.)

\( \text{COV}: \)  coefficient of variation.

\( d: \)  the beam effective depth (in.)

\( d_b: \)  the developed/spliced bar nominal diameter (in.)

\( d_s: \)  the nominal diameter of the stirrup (in.)

\( E_c: \)  concrete elastic modulus (ksi)

\( E_s: \)  steel elastic modulus (ksi)

\( f'_{c}: \)  concrete compressive strength (psi)
\( f_i: \) the bar stress at the location of the strain gage \((i)\).

\( f_{i-1}: \) the bar stress at the location of the next strain gage \((i-1)\).

\( f_y: \) yield strength of the bar \((\text{psi})\)

\( f_{y_t}: \) the yield strength of the transverse reinforcement \((\text{psi})\)

\( h: \) beam height \((\text{in.})\)

\( j_{d1}, j_{d2}: \) level arms of bending moments in two successive cracked sections \((\text{in.})\)

\( K_1: \) a constant

\( K_{tr}: \) term representing the effect of transverse reinforcement on bond strength.

\( l: \) beam length \((\text{in.})\)

\( l_s: \) splice length \((\text{in.})\)

\( l_d: \) development length \((\text{in.})\)

\( \Delta L: \) distance along the beam length between the two considered section.

\( M_1, M_2 \) bending moments at two successive cracked sections \(\text{(kips-in.)}\)

\( N: \) total number of stirrups or hoops along the splice/development length.

\( n: \) number of spliced/developed bars in the section.

\( P: \) total applied load at bond failure \((\text{kips})\).

\( R^2: \) mean square error.

\( R_r: \) the relative rib area of the bar \((\text{ratio of projected rib area normal to bar axis to the product of the nominal bar perimeter and the centerline -to-centerline rib spacing})\)

\( s: \) average spacing of the transverse reinforcement \((\text{in.})\)

\( T_b: \) total force in the bar at bond failure \((\text{lb.})\)

\( T_c: \) concrete contribution to total force in a bar at bond failure \((\text{lb.})\)

\( T_s: \) steel contribution to total force in a bar at bond failure \((\text{lb.})\)
$t_d$: term representing the effect of bar size on $T_s$. The values vary according to the design expression in which it is used. $t_d = 0.72 \ d_b + 0.28$ used in expression developed by Darwin et al. [1996], and $t_d = 0.78 \ d_b + 0.22$ used in expression developed by Zuo and Darwin [2000].

$t_r$: term representing the effect of bar size on $T_s$. $t_r = 9.6 \ R_r + 0.28$

$\Delta T$: the change in the bar tensile force (kips).

$u_{\text{av}}$: the average bond stress (ksi)

$u_{bij}$: the average bond stress between strain gage $(i)$ and $(i-1)$

$x_i$: the distance between the loaded end and the strain gage $(i)$.

$x_{i-1}$: the distance between the loaded end and the strain gage $(i-1)$.

$\alpha$: reinforcement location factor; (top cast or bottom cast)

$\beta$: coating factor (coated or uncoated bars)

$\lambda$: light weight aggregate factor

$\gamma$: reinforcement size factor ( =0.8 for # 6 bars and smaller, and = 1.0 for otherwise)

$\Phi_b$: overall strength reduction factor against bond failure.

$x_i$: the distance between the loaded end and the strain gage $(i)$. 
1.1 General

Corrosion problem of steel reinforcing bars embedded in reinforced concrete structures is one of the most widely spread, costly structural problems that face reinforced concrete structures located in aggressive environments. The potential consequences of the corrosion problem can be summed up in the continuous reduction in strength, stiffness, durability, and designed life time of concrete structural elements reinforced with conventional steel. Considerable percentages of the nation’s national budget are assigned either for innovative research works that can come up with radical solutions for the corrosion problem or for repair, strengthen, and in some cases reconstruct damaged concrete structures. During the last several decades, researchers succeeded in developing and implementing quite few effective methods that can either restrain the corrosion process in already infected structures or prevent the initiation of corrosion in healthy structures.

1.2 Corrosion Mechanism

Corrosion of reinforcing steel bars embedded in concrete members is an electrochemical process that involves the decay of the steel rebars with time due to the formation of an electrochemical galvanic cell within the concrete member between an anode, represented by the steel, and a cathode represented by the concrete. The formation of such galvanic cell is directly influenced by the presence of either chloride or carbon dioxide ions and by the richness of the surrounding media with both moisture and oxygen. This corrosion
process is normally accompanied by the formation of the rust as a byproduct.

Concrete, normally, offers a highly alkaline corrosion-protection environment to the embedded reinforcing steel (pH value ranges between 12-13). The high alkalinity of the concrete is due to the presence of the calcium, sodium, and potassium hydroxide ions in the cement paste. Such high alkalinity creates a thin protective passive layer made of iron oxide that surrounds the surface of the embedded steel bar. This iron oxide film acts as a barrier that isolates the steel from the surrounding media and thus protects the steel from further corrosion. However, when this protective oxide film is attacked by chloride ions or carbon dioxide ions, it will start to decay and chemically decompose exposing the steel to the surrounding aggressive environment and corrosion process will start. The corrosive environments are those environments characterized by having either high chloride ion contents, as seawater, deicing salts, and salt spray or high acidity content, as carbon dioxide and sulphate ions. The presence of adequate oxygen diffusion and continuous moisture supply is vital to ensure the continuity of the corrosion process.

1.3 Corrosion Vulnerability

Worldwide, billions of dollars are directed, annually, to serve the corrosion problem. Around 3% of the General National Product (GNP) of each country around the world is dedicated to protect, repair, strengthen and in some cases reconstruct deteriorated structures. The cost of corrosion to U.S. industries and the American public is currently estimated by the U.S. Congress to be $276 billion per year, and according to the Federal Highway Association, the infrastructure deficiencies due to corrosion present a $1.3 trillion problem
The magnitude of the problem in the transportation infrastructure has increased significantly in the last three decades and is likely to keep increasing. According to a 1997 report out of the 581,862 bridges in and off the U.S. federal-aid system, about 101,518 bridges were rated as structurally deficient. Most of these bridges were not in danger of collapse, but they were likely to be load posted so that overweight trucks will be required to take a longer alternative route.

1.4 Corrosion Damages

The key reason for structural deterioration in salt contaminated concrete structures is the rust formation. The rust normally tends to occupy a volume of about two to three times that of the parent steel. Being present in a confined space, it is likely to exert as high as 5000 psi internal outward pressure on the surrounding concrete cover, Elsener [2001]. Such internal pressure is high enough to overcome the tensile strength of concrete causing cracking, spalling, de-lamination and progressive deterioration of the concrete cover with time.

Bridge and parking garages, marine and underground structures, highway pavements, and other structures that are continuously exposed to deicing salts, seawater or salt chloride environments are the most popular construction suffering from the corrosion. Such structures suffer from a series of structural, durability and performance problems. The structural problems are related to the problem of concrete cover loss and reinforcing steel mass loss. The concrete cover loss will lead to reduction in the concrete cross sectional area causing
significant loss of bond strength followed by consequent reduction in element stiffness and strength. The reduction in the element stiffness and strength will be pushed further by the effect of the reduction in the cross sectional area of the reinforcing bars. As a result of such stiffness degradation, excessive deformations and deflections, beyond the pre-designed values, are expected to be seen. Such undesirable and excessive deformations are the main source of the serviceability deficiencies and the structure malfunction. Durability problems are also expected leading to premature deterioration of concrete matrix followed by an overall reduction in the structure designed life time.

1.5 Methods of Corrosion Resistance

There are two major measures, namely active and passive, that have been developed and successfully implemented to control, minimize and in some cases eliminate the corrosion risk. The passive measurements are designed to be used for protecting the healthy concrete elements that suffer no corrosion problems against potential corrosion attack. These passive measurements include using high performance concrete with added chemical admixtures, as super-plasticizers, or mineral admixtures, as fly ash and silica fumes, or using corrosion inhibitors that can improve the passivation stability of the steel, or through using epoxy coated or galvanized re-bars or applying sealers to the finished concrete surface. On the other hand, the active corrosion control systems are assigned to treat the salt contaminated concrete elements by putting an end to the corrosion process or at least minimizing the rate of corrosion; this can only be done using cathodic protection, Elsener [2001].
1.6 MMFX

In 1998, MMFX Technologies Corporation announced that they were able to produce a different and unique type of high strength reinforcing steel bars. They named their new product MMFX Steel (Micro-composite Multi-structural Formable Steel). The company’s brochure claimed that their new steel product is unique because it has a significant high corrosion resistance compared to the corrosion resistance offered by the conventional carbon steel A615 currently used. They added that these newly invented steel rebars are at least equal, or in many cases, far superior to the existing conventional steel rebars in their mechanical properties of strength, ductility, toughness, brittleness and formability. They also claimed that the MMFX steel is economical to produce.

The corrosion resistance and mechanical properties of the MMFX steel is referred to the MMFX patent and proprietary technology achieved through developing an optimum micro structure composition of the steel. The proprietary material composition and production processes minimize the formation of micro galvanic cells and delay the initiation of corrosion. The MMFX steel initial proprietary technology was developed at the National Center for Electron Microscopy at the Lawrence Berkeley National Laboratory under the guidance of Professor Gareth Thomas. After about 25 years of research and development, the research team was able to design steel that had a totally different microstructure. This fundamental change to the microstructure composition produced steel with significant high corrosion resistance and superior mechanical properties.

MMFX Technologies Corporation is the name of the corporation founded in 1998 to commercialize the proprietary, micro and nano technology. MMFX operates as a
parent/subsidiary corporate structure. It currently has two operating subsidiaries – MMFX Steel Corporation of America and Fasteel – whose missions are to commercialize MMFX products, produce and sell rebars.

1.7 Micro Structure of MMFX

The development of these technologies was made possible by the aid of the atomic resolution capabilities offered by modern electron microscopy. The electron microscope actually enables the scientist to observe and closely inspect a material’s micro structure. Using this unique tool, Professor Thomas developed an understanding of steel technology down to the atomic level, and managed to manipulate the mechanical properties of materials and obtain, an optimum microstructural properties

1.8 How is the MMFX Composition Different

Typical carbon steels currently used (A615, A706) have a ferritic-pearlitic microstructure, in which carbides form in the iron carbide phase, see figures (1.1) and (1.2). An electrochemical reaction takes place between the ferrite (Anodic) and iron-carbide (cathodic) phase. Carbide particles form galvanic cells by their contact with the iron matrix (ferrite), see figure (1.3). Such galvanic cells in corrosive environments become micro-galvanic cells – the driving force behind corrosion. Micro galvanic cells, which are inherent to this microstructure, lead to a chemical reaction resulting in the formation of ferrous oxides or rust. Thus by eliminating or minimizing the formation of carbides in the steel microstructure, corrosion activity can either be eliminated or minimized. The company’s
patented proprietary steel technology forms a matrix that is almost carbide free. This unique physical feature minimizes the formation of micro-galvanic cells hence reduces corrosion initiation. These cells and the fibers they form; significantly initiate and speed up corrosive activity in the steel. In a moist environment, a battery-like effect occurs between the carbides and the ferrites that destroy the steel from the inside out. This micro-galvanic cell effect is the primary corrosion initiator that drives the corrosion reaction, MMFX product Bulletin [2002].

MMFX steel has a completely different structure at the nano or atomic scale; a laminated lath structure resembling plywood, refer to figure (1.4). Steel made using MMFX nanotechnology does not form micro-galvanic cells. MMFX’s “plywood” effect lends amazing strength, ductility, toughness and corrosion resistance. Most steel exhibits strength at the cost of ductility or brittleness. Steel, made using MMFX’s proprietary technology is not only stronger and tougher, but is also significantly more corrosion resistant than conventional steel.

The critical chloride threshold and corrosion rate for different steel reinforcement types including MMFX steel was evaluated at the Texas Engineering Experimental Station (TEES) at Texas A&M University. They reported that average chloride threshold, in lbs of chloride ions per cubic yard of concrete, for the MMFX steel rebars was 8.8 times that of the conventional carbon steel A615 when tested in accordance with Patent Pending ACT (Accelerated Chloride Threshold) test procedure, Trejo [2002]. The time to corrosion initiation for MMFX steel rebars is 69 years compared to 22 years for the carbon A615 steel. Figures (1.5), (1.6) and (1.7) demonstrate that MMFX steel has little tendency to corrode in
comparison to A615 steel when exposed to aggressive corrosive environment, in this case a highly saline solution.

The basic difference between the MMFX and the A615 carbon steel is in the

1. Chemical Composition; elements incorporated in its chemical composition.
2. Proportions (percentages) of the basic elements.
3. The distribution pattern of such elements within the cross section on the micro-structure level.
4. Production technology implemented in its production.

### 1.8.1 Chemical Composition:

As stated by the owner company, the chemical composition of the MMFX steel consists mainly of iron, chrome, and carbon atoms, the MMFX steel is characterized by its considerably low carbon content, less than 1%, and around from 8-10% chrome. The company claims that the negligible amount of nickel, present in the composition, makes this steel economical to produce. Table (1.1) shows the percentage of the different elements forming the MMFX steel by weight.

### 1.9 Potential Applications

The high corrosion resistance and the superior mechanical properties of the MMFX steel rebar allow it to be widely used in a significant number of field applications. Specially, in structural elements which are continuously exposed to chlorides or carbonation or other kinds of corrosive environments. The high strength properties can be useful in reducing the
reinforcement ratios in high reinforced members. This will ease the process of concrete casting and improve the vibrating and shorten the construction time and effort of steel cages. Deicing salt exposed structures as bridge and parking decks and garages, as well as sea water exposed structures like marine and shore structures are the potential future application for this new steel.

1.10 Advantages and Disadvantages of the Current Available Alternatives

Presently, there are quite few alternative rebars that are currently used to minimize the corrosion problem in reinforced concrete structures exposed to sever environments.

1.10.1 The epoxy coated steel:

Advantages:

In some environments epoxy coat rebar may outperform ASTM A615 rebar, adding a reported 5 years to the service life of a structure.

Disadvantages:

a. The substrate material (A615) is not corrosion resistant and depends on the epoxy coating for the bar’s corrosion resistance.

b. Field fabrication for epoxy-coated rebar is difficult because the coating needs to be placed on the bar after fabrication/bending and prior to shipment for installation.

c. Damage to the coating (i.e. nicks, scratches or holidays) often accelerates the initiation of corrosion at these locations. This is because of localized corrosion at the damaged surface
can lead to under film corrosion.

d. Field cutting of the material requires field coating as part of its installation process, adding to additional inspection and installation costs.

e. The epoxy coatings can de-bond from the substrate material leading to coating cracking and damage after placement.

f. Epoxy coating requires use of knockdown design factors to take into account bond/shear loss due to the coating.

1.10.2 Stainless Steel Clad (A615):

Advantages:

a. Stainless steel cladding provides this rebar product a relatively corrosion resistant material.

Disadvantages:

a. The substrate material (A615) is not corrosion resistant and depends on the clad coating for its corrosion resistance.

b. Clad thickness tends to be non-uniform especially in smaller rebar sizes (#4 and #5 bars.)

c. Bending of smaller bars sizes has been reported to result in splitting, cracking or de-bonding of cladding.

d. All exposed bar ends need to be capped to prevent exposure of the substrate material to corrosive environments.

e. Field cutting of bar may require caps to be field placed adding to inspection and installation costs.
f. Standard 40-foot rebar lengths in many cases may not be available, requiring additional field splicing.

1.10.3 *Stainless Steel:*

**Advantages:**

a. Stainless steel rebar has excellent corrosion resistance when mill scale is removed.

**Disadvantages:**

a. Stainless steel rebar has an estimated installed cost of from 4 ½ to 5 1/2 times that of conventional (A615) rebar.

b. These steels generally have lower yield strengths than carbon steels, requiring additional steel in comparison to A615.

c. Standard 40-foot rebar lengths in many cases may not be available requiring additional field splicing.

1.10.4 *MMFX Steel:*

**Advantages:**

a. MMFX is highly corrosion resistant without use of coating technologies, as a result of its patented chemical composition and microstructure.

b. MMFX’s high design yield strength (110 ksi) allows for design reduction of rebar quantities and simplifying field installation of the rebar.

c. Standard fabrication techniques can be used at steel fabricators and in the field.
Disadvantages:

a. MMFX Steel is moderately more expensive than A615, but more economical when used effectively.

1.11 Research Significance

For any reinforced concrete element to structurally function there should exist a sufficient bond between the concrete and the reinforcing bars to allow for force flow between the two materials. The significance of this research appears from the ability of the study to examine the bond properties of the MMFX micro-composite steel with concrete. The importance of conducting an experimental investigation to identify the bond characteristic of this new rebars with concrete is to allow for safe, reliable and efficient utilization of the new MMFX rebars. The full understanding of the behavior of this steel with concrete will encourage the confident implementation of the MMFX steel rebar in the field by engineering community.

1.12 Research Scope and Objectives

The objective of this experimental investigation is to examine the bond properties, including the bond strength, of the MMFX steel rebars with concrete. The investigation focused on studying the bond of deformed, straight, unanchored, MMFX rebars with concrete in flexural members and examining the applicability of the current ACI code design expression and some other reliable and accurate descriptive equations to the MMFX steel rebars. In this regard, twelve specimens were prepared and tested. Four of them were beam
end specimens and eight were full scale splice specimens. For the beam end specimens; the parameters considered were bar location and amount of transverse reinforcement confining the bonded length. However, for the splice specimens, the bar size and the splice length were the studied parameters. The bond behavior was observed and recorded and analyzed from the measured data in all the specimens. The experimental bond strength was compared to the predicted values using the most reliable and accurate equations. Conclusions were made in the light of the test results and recommendations were presented.
Figure (1.1) Micro Structural Composition of Conventional Steel

Figure (1.2) Conventional Steel Micrograph

Figure (1.3) Micro-galvanic Cell Formation between Iron-carbide and Ferrite Phases

Anodic Reaction:
Fe → Fe^{2+} + 2e^-

Cathodic Reaction:
O_2 + H_2O + 4e^- → 4OH^-  
Fe^{2+} + 2OH^- → Fe(OH)_2
Figure (1.4) MMFX2 - Microstructure

Untransformed Nano Sheets of Austenite

Prior austenite grain boundary

Dislocated Laths martensite

Figure (1.5) Comparison between the MMFX Steel and A615 Steel Regarding Degree of Corrosion

<table>
<thead>
<tr>
<th>ASTM A615 Conventional Carbon Steel Rebars</th>
<th>MMFX Steel</th>
</tr>
</thead>
</table>

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Figure (1.6) A615 to the Left compared to MMFX steel to the Right, picture taken in Aug. 7, 2000

Figure (1.7) A615 to the Left compared to MMFX Steel to the Right, picture taken in Aug. 25, 2000
Table (1.1) the percentage of the major elements forming the MMFX steel by weight.

<table>
<thead>
<tr>
<th>Element</th>
<th>Carbon</th>
<th>Chromium</th>
<th>Manganese</th>
<th>Nitrogen</th>
<th>Phosphorus</th>
<th>Sulfur</th>
<th>Silicon</th>
</tr>
</thead>
<tbody>
<tr>
<td>Amount</td>
<td>0.10%</td>
<td>6 to 11%</td>
<td>1.5%</td>
<td>400 ppm</td>
<td>0.02%</td>
<td>0.025%</td>
<td>0.50%</td>
</tr>
</tbody>
</table>
2.1 General

In this chapter a brief discussion about the bond characteristics of steel rebar and the concrete is presented. This review will underline the importance of the presence of bond between concrete and rebars, bond force definition, and mechanisms of bond transfer in both plain and deformed bars. Also, the different modes of bond failure and the most commonly used configurations of bond specimens are presented. Later in this chapter, some of the major parameters that have significant influence on the bond properties are discussed. Before the end of this chapter, the past research works that have been used as reference for this research program and some important descriptive bond equations and currently available design equations for bond are discussed. Comparison of the draft of the ACI committee 408 and current code approach are also provided to highlight their reliability and accuracy. Mechanical properties and composite behavior of the MMFX rebars with concrete is also highlighted in this chapter.

2.2 Bond Significance

Reinforcing bars used to reinforce structural concrete members are very beneficial as they enhance ductility of the concrete member, assure better utilization of the concrete section, and increase the load capacity of the member. The basic idea of using the reinforcing bars in a concrete section in flexural members is to transfer the tensile forces across the tension cracks. Such state of force transfer requires adequate bond between the two elements.
Thus, in case of straight unhooked bars, the existence of a reliable mechanism of force transfer between the reinforcing bars and the surrounding concrete is the only way to achieve composite action between the two materials and to ensure that they will structurally act together as one unit. In order to achieve the needed bond, there are more than one mechanism to establish a force transfer between the two materials; such as friction and mechanical bearing which will be discussed later.

The role played by the reinforcing steel in concrete elements varies with the static system of the element and the pattern of the internal stress distribution. For example; in cracked flexural members along a constant moment region, the tensile stresses developed in the concrete section are gradually transferred to the reinforcing bar along a development length. The bar carries the tensile forces across the crack to the concrete section on the other side of the crack. And through an anchorage length, the tensile force in the bar is retransferred back to the surrounding concrete. On the other hand, between any two successive cracks, in a constant moment region, the tensile forces accumulated in the rebar do not flow back to the concrete; however, these forces continue to flow through the bar until they reach the next crack. But the presence of bond between the two materials will cause an in and out flow of force from the rebar to the surrounding concrete and back again causing a phenomenon known as tension stiffening as shown in figure (2.1). This same phenomenon of tension stiffening could be observed in reinforced concrete tension members between any two successive cracks. In cracked flexural members, along the shear span region, and between any two successive cracks, part of the tensile forces carried by the bar has to be transferred to the surrounding concrete through bond along the distance between the two cracks as shown in figure (2.2).
2.3 Bond Stresses

Bond forces in reinforced concrete structures are the longitudinal shearing forces per unit length of the bar that develops on the interface of the reinforcing bar and the surrounding concrete matrix. Therefore, the bond force can be defined as the rate of change of axial force in the reinforcing bar with the bar length. Bond force distribution along the bonded length of tensioned bar is, generally, not uniform. Bond stresses can be defined as bond force per unit surface area of the bar. Average bond stresses \( u_{av} \) along a specified length \( \Delta L \) can be computed from the following relation:

\[
u_{av} = \frac{\Delta T}{\Delta L \pi d_b}
\]

Equation (2.1)

Where:

\( u_{av} \): is the average bond stresses along a definite length \( \Delta L \).

\( \Delta T = (T2 - T1) \): is the change in the bar tensile force along a distance \( \Delta L \).

\( \Delta L \): is the distance along the beam length between the two considered section.

\( d_b \): is the nominal bar diameter.

Along the shear span, in flexural reinforced concrete members, the variation in the bar tensile force is a result of the variation in the magnitude of the inducing bending moment, see figure (2.3). Thus the bond stresses given in equation (2.1) can be rewritten in terms of the bending moments at each section (M1 and M2) and the corresponding internal lever arm.
(jd1 and jd2) in the following format:

\[ u_{av} = \frac{M_2 - M_1}{jld_2 - jld_1} \frac{1}{\Delta L \pi d_p} \]  

Equation (2.2)

2.4 Bond Mechanism in Plain and Deformed Bars

It is worth to be noted that despite the wide usage of deformed steel bars in structural applications, the study of bond behavior and properties of such bars is, in fact, rather more sophisticated than plain bars. In plain bars, without surface deformations, creation of bond stresses is mainly due to the presence of two mechanisms of stress transfer; namely the chemical adhesion between surface particles of the materials and the surface friction. The stresses that can flow in-between the two materials through the influence of chemical adhesion are considerably low as the capacity of the adhesion force between the two materials is relatively small and is very sensitive to any relative movement between the bar and the surrounding concrete. The friction created between the two materials’ surfaces is basically the main source of stress convey. Friction stresses are developed due to the roughness of concrete mortar surface, the slight irregularities and unevenness along the plain bar surface, together with the concrete shrinkage.

On the other hand, deformed bars were designed to have a different load transfer mechanism than plain bars. In case of deformed bars, the process of force transfer from one material to the other relies on presence of three major systems; the chemical adhesion and the friction, previously mentioned, and the third system is through the mechanical interlocking or
direct bearing of the rebar ribs on the concrete paste surrounding the bar.

The relative longitudinal movement of the rebar with respect to the surrounding concrete is known as the bar slip. When a deformed bar starts to slip, surface adhesion forces are significantly lost. Such slip activates both the bearing forces on the ribs and the friction forces in between the ribs. As slip increases, the friction forces are also obviously reduced, leaving most of the bond stresses to be transferred through direct bearing. Thus, the bearing stresses on the bar ribs become the principal mechanism of force transfer, see figure (2.4). Since the face of any bar rib is, normally, inclined to the bar longitudinal axis with an angle less than $90^0$, the bearing of the bar ribs on the concrete keys, entrapped in between the bar ribs, results in an inclined bearing force. Such force can be analyzed into two components; a longitudinal component, parallel to the bar axis which represents the bond force and a radial component which causes an outward radial pressure on the surrounding concrete. Such effect is known as the wedge action.

2.5 Modes of Bond Failure:

Two modes of bond failure are typically observed when dealing with deformed bars, which are known as:

a) Splitting bond failure; which results from the splitting of the smallest concrete cover surrounding the bar and is most likely to occur in cases of low levels of confinement around the bonded bar. Cases of relatively small concrete covers and/or small amount of transverse reinforcement represent state of low degree of confinement, see figure (2.5). This phenomenon, of cover splitting, happens when the tangential tensile stresses in the concrete
cover, caused by the radial outward pressure, exceed the tensile strength of the concrete as shown in figure (2.6).

b) and Pullout failure; which results from the crushing and shearing off of the concrete keys enclosed between the bar ribs along a cylindrical plan of failure which passes by the bar rib tips, see figure (2.7).

In the pullout failure, the bar will slide along a shearing cylindrical plane of failure keeping the concrete in between the ribs. This mode of bond failure is more likely to occur when the level of the radial confinement surrounding the anchored bar, caused by the presence of relatively thick concrete covers and/or excessive transverse stirrups, is high enough to delay the splitting mode of failure. Development/anchorage length of a bar can be defined as the minimum length needed to develop/transfer yielding stresses in/from the bar from/to the surrounding concrete.

2.6 Types of Bond Specimens:

A number of bond test specimen configuration have been used to investigate the bond behavior between reinforcing bars and concrete. There are four major and most commonly used configurations; pullout specimens, beam end specimens, beam anchorage specimens, and splice specimens. The nature of both, bond response and bond strength, is affected by the variation in bond specimen configuration. In the coming discussion, the description of the configuration and the advantages and the disadvantages of each type will be briefly discussed.
2.6.1 **Pullout specimens:**

Because of their ease of fabrication and simplicity in testing, pullout test specimens like those shown in figures (2.8a and 2.8b) have been chosen by many researchers to evaluate bond performance of reinforcing bars embedded in concrete. Generally, the bars were pulled from the surrounding concrete in such a way that the concrete surrounding the bar is subjected to compression. At the same time, compression struts will form between the bar surface and the loading points on the concrete surface. In actual concrete constructions, such state of stresses is unlikely to happen. Normally, in reinforced concrete elements, both the bar and the surrounding concrete are placed in tension.

Such transverse compression struts have a positive effect on increasing the bond strength and are not simulating typical situations encountered in structures and bridges. That is why this type of bond specimens is not recommended by the ACI Committee 408 to be used to determine the development length as they represent the least realistic type of the four types of bond. As a result, various configurations of test specimens have been proposed by many researchers to eliminate transverse compression as shown in Figures (2.8c) through (2.8e).

2.6.2 **Beam End Specimens:**

The beam end specimens, shown in Figure (2.8e) represent a more realistic type of specimens that can give better and more accurate results for bond behavior than pullout specimens. In the beam end specimens, both the reinforcing and the surrounding concrete are subjected to tensile stresses. To achieve such state of stresses, the compressive forces must be
located away from the reinforcing bar by a distance not less than the bonded length of the tested bar. Also a short length of the tested bar near the concrete free surface has to be unbonded from surrounding concrete to avoid conical failure. The ASTM standard A944-99, established to compare the bond strength of steel reinforcing bars to concrete, adopts this type of bond specimens. It was experimentally found that the bond results obtained using the beam end specimens closely match those obtained using full-scale reinforced concrete members.

2.6.3 Beam Anchorage and Splice Specimens:

They represent full scale specimens, designed to directly measure the bond strength of development/spliced bars. Those specimens replicate a flexural member with a defined bond length. Splice specimens are normally designed and tested under four point loading having the splice length lying within the constant moment region. The ease of their fabrication and the close similarity between the stress profile, in both the concrete and rebar, and the actual stress profile in real flexural members entitles the splice specimens to be the main source of the experimental data used to establish the current design provisions for development length.

2.7 Major Parameters Affecting Bond Behavior

In general, many parameters influence the bond properties of reinforcing bars to concrete. From the most significant factors; thickness of the concrete covers (bottom and/or side), bar spacing, development and splice length, amount of confining reinforcement, and
concrete strength. In general, bond forces increase with the increase in thickness of the concrete covers (bottom and/or side), bar spacing, development and splice length, and amount of confining reinforcement. Top cast bars have lower bond strength than bottom cast bars. The bond force, for a given length, mobilized by both concrete and transverse reinforcement increases as the bar diameter increases. Also the bond strength of bars confined by transverse reinforcement will increase with the increase in the relative rib area. Bond strength increases with increasing concrete compressive strength for bars not confined by transverse reinforcement approximately with the (¼) power of the compressive strength ($f_c^{\frac{1}{4}}$). The additional bond strength; provided by transverse reinforcing, increases approximately with the (¼) power of the compressive strength ($f_c^{\frac{1}{4}}$). An increase in the aggregate quantity and strength results also in an increase in bond strength. In the experimental investigation to study the bond behavior of the MMFX steel rebars, four of these major parameters were considered; namely bar size, amount transverse reinforcement, and bonded length, and bar cast position. A detailed summary on the impact of the chosen parameters on the bond capacity and characteristics is presented in the following section.

2.7.1 Development/Splice Length:

Increasing the development/splice length of a reinforcing bar, in general, will increase its bond capacity. However, the nature of the bond failure results in a non-proportional increase in the bond strength with the increase of bonded length. The explanation of this fact is that the distribution of bond forces is not uniform along the development length and that bond failures tend to be incremental, starting in the region of the highest bond force per unit
length. In case of anchorage bars, longitudinal splitting of the concrete initiate at either a free surface or a transverse flexural crack where the bar will have the highest stress. For spliced bars, splitting starts at the ends of the splice, moving towards the center of the splice. For normal strength concrete, crushing of concrete in front of the ribs is likely to occur. For spliced specimens studied after failure, it is common to see no crushed concrete at ribs near the tension end of the spliced bar, with the crushed concrete located at the end of the bar, indicating that the failure occurred by a slow wedging action followed by rapid final movement of the bar at failure. Because of the mode of bond failure, the unloaded end of a developed length is less effective than the loaded end in transferring bond forces, explaining the non proportional relationship between development/splice length and bond strength.

Although the relationship between bond force and development/splice length is not proportional, it is nearly linear. When failure occurs, a significant crack area is opened in the member due to splitting, Tholen and Darwin [1996]. As the embedment length of the bar increases, the cracked surface at failure also increases in a linear but not proportional manner with respect to the development/splice length. Thus, the total energy needed to form the crack and, in turn, the total bond force required to fail the member, increases at a rate that is less than the increase in bonded length. Therefore, the common design practice [ACI 318-02] of establishing a proportional relationship between bond force and development/splice length is highly conservative for very short bonded lengths, but becomes progressively less conservative, and eventually unconservative, as the bonded length and stress in the developed/spliced bar increase.
2.7.2 Transverse Reinforcement:

The presence of transverse reinforcement as a confinement surrounding the developed/spliced bars will increase the ultimate bond force by delaying the progression of splitting cracks. Thus increasing the magnitude of the bond force needed to cause splitting bond failure. At the same time, the magnitude of the bond force needed to cause pullout bond failure remains unaltered as it is almost not affected by the increase in the transverse confinement. According to that, the excessive increase in transverse reinforcement will eventually convert the mode of bond failure from splitting to pullout failure. Any additional confinement, more than that needed to cause the change from a splitting to a pullout failure, becomes gradually non effective and eventually provides no increase in bond strength, Orangun et al. [1977]. The total bond force of a developed/spliced bar $T_b$ can be represented as the sum of concrete contribution $T_c$, representing the bond force that would be developed without the transverse reinforcement, plus a steel contribution $T_s$, representing the additional bond strength provided by the transverse reinforcement.

$$T_b = T_c + T_s$$  \hspace{1cm} \text{Equation (2.3)}

The value of the concrete contribution, $T_c$, is affected somewhat by the presence of the transverse steel, because the effective crack length between bars is reduced as bar slip continues in the process of mobilizing the additional bond strength provided by the transverse reinforcement, Zuo and Darwin [1998, 2000]. The effect of transverse reinforcement on $T_c$ is small but measurable. The value of the steel contribution $T_s$ is a
function of the area of reinforcing steel that crosses potential crack planes, the strength of the concrete, and both the size and deformation properties of the developed/spliced reinforcement. \( T_s \) can be represented in the following format:

\[
T_s = K_f t_r t_d \frac{N A_{tr}}{n} f_c''^p
\]

Where;

\( K_f \): is a constant.

\( t_r \): is a factor that depends on the relative rib area \((R_r)\) of the reinforcement.

\( t_d \): is a factor that depends on the diameter \((d_b)\) of the developed/spliced bar.

\( N \): the number of transverse stirrups, or ties, within the development/splice length.

\( A_{tr} \): area of each stirrup or tie crossing the potential plan of splitting adjacent to the reinforcement being developed or spliced.

\( n \): number of bars being developed/spliced along the plane of splitting.

\( f_c'' \): concrete compressive strength based on 6 in. x 12 in. cylinders.

\( p \): power of \( f_c'' \) between 0.75 and 1.00.

It was observed by Zuo and Darwin [1998, 2000] that an increase in the wedging action of the bars, resulting from both the increase in \( R_r \), a relative measure of rib size and spacing, and the increase in bar size \( d_b \), will increase the stress in the stirrups, resulting in an increase in confining force. The relationship between confinement and the degree of wedging action is in concert with the observation that stirrups rarely yield, thus allowing an increase in lateral displacement to be translated into an increase in confining force Sakurada et al.
30. As a result, the yield strength of the transverse reinforcement does not play a role in the steel contribution to bond force $T_s$.

2.7.3 **Bar Size:**

The relationship between bar size and bond strength is not always appreciated because: (1) A longer development/splice length will always be required as the bar size increases in order to achieve the same yielding stress in the bar; (2) For a given bonded length, larger bars in diameter achieve higher total bond forces than smaller bars for the same level of transverse confinement.

Considering the second issue first, for a given bonded length, large size bars require large forces to cause either splitting or pullout failure due to their large perimeter and surface area. The result is that the total bond force, developed at bond failure, is not only an increasing function of concrete cover, bar spacing, and development/splice length, but also of bar cross sectional area, Orangun et al. [1977]; Darwin et al. [1992, 1996b]. The bond force at failure, however, is not proportional to the bar area, which means that a longer embedment length is needed for a larger bar to fully develop a given bar stress. In terms of bond stresses, smaller bars have a greater advantage. Thus conventional wisdom suggests using a larger number of smaller bars rather than a smaller number of larger bars.

The size of a developed bar also plays an important role in the contribution of confining transverse reinforcement to bond strength. As larger bars slip, higher strains and thus higher stresses are activated in the transverse reinforcement, providing better confinement. As a result the added bond strength provided by transverse reinforcement...
increases as the size of the bar increases.

2.7.4 Bar Location at Casting:

Top cast bars have lower bond strength than bars cast near the bottom of the specimens, Ferguson and Thompson [1962]. The lower bond strength of top-cast bars can be explained based on the negative effect caused by the settlement and bleeding of the concrete on the concrete strength. Thus an increase in the concrete depth below the top-cast bar leads to reduction in bond strength. It was observed that, for top cast bars; the rate of reduction in the bond strength with concrete cover thickness is significantly higher than that for bottom cast bars, Netherlands [CUR 1963].

2.7.5 Steel Stress and yield Strength:

Orangun et al. [1975] reported that yielding of developed/spliced bars will cause reduction in bond strength. Darwin et al. [1995b], [1996b] and Zuo and Darwin [2000] reached an opposite conclusion that the bond strength for bars that yield before failure is generally higher than that of higher strength bars having the same bonded length and same confinement but do not yield before failure. Darwin et al. [1995b], [1996b] observed that, in case of bars not confined by transverse reinforcement, the yielding of the spliced bars has no effect on the bond strength.

2.8 Reported Research Work:

Clark [1949] tested deformed bars in both beams and pull-out specimens, considering
the bar size, bar’s rib deformation pattern, concrete strength and development length as test variables. The objective was to determine the correlation between bond strength values obtained from beam and pull-out specimens for each of the above mentioned parameters. The author pointed out that the correlation between the results of the beam and pullout specimens is strong enough to give reliable estimates of the bonding efficiency of deformed reinforcing bars, and added that both the load-bond slip relations and the general behavior of the bars were similar in the two types of tests.

Mathey et al. [1961] investigated the bond strength of high yield strength deformed bars in beam and pull-out specimens by varying the bar diameter, and anchorage length. The authors adopted a new bond failure criterion in which he defined bond failure as a failure accompanied by excessive slip at the free end of the bar with only a slight increase in the bar force. The critical bond stress was defined to be the lesser of the bond stresses corresponding to either a free-end slip of 0.002 in. or loaded-end slip of 0.01 in.

Ferguson and Thompson [1965] focused on the development length of large sized, #11, high strength bars, \( f_y > 75 \text{ ksi} \). The concrete cover, beam width, stirrup ratio, and depth of concrete cast below the bar were the main variables. It was found that increasing concrete clear cover surrounding the bar generally increases ultimate bond strength with a rate of 60 psi/in of cover, but it did not improve the observed crack width on the surface under service loads. Also the beam width and shear were reported as factors affecting bond strength and that any further increase in development lengths more than 50 in will no longer cause decrease in bond strength.
Ingraffea et al. [1984] reported that, in general, there are four different sources that contribute to bond slip namely elastic deformation, crushing of concrete at points where concrete bears on steel ribs, secondary radial cracking, and longitudinal splitting cracking.

The splice full scale beam bond specimens were selected by a group of researchers, Zuo and Darwin [2000], Chinn et al. [1955], Hester et al. [1993], Ferguson and Breen [1965], Darwin et al. [1996], Azizinamini et al. [1995], Azizinamini et al. [1993], Chamberlin [1958], and Rezansoff et al. [1993] to run their bond investigations. The specimens used by Zuo and Darwin [2000] and Darwin et al. [1996] were 16 ft long, however, Azizinamini et al. [1993] chose 20 ft long specimens. The nominal width for the tested specimens was either 12 or 18 in. and the nominal height ranged between 15 and 18 in. Zuo and Darwin [2000], Darwin et al. [1996], and Hester et al. [1993] tested #8 and #11 bars representing the large size bars and also used #5 and #6 to represent the smaller bars. Although all the above mentioned researchers tested beams with two spliced bars in the section, however Hester et al. [1993] were the only ones to test beams with one spliced bar. Zuo and Darwin [2000], Azizinamini et al. [1995], Rezansoff et al. [1993], and Hester et al. [1993] used, in some of their tested beams, #3 grade 60 closed stirrups as a transverse reinforcement to confine the development/splice length of the tested bar. In other beams, they used no stirrups at all in the splice region. Rezansoff et al. [1993] was the only researcher, from the above mentioned, to install strain gages on the tested bar surface to measure the steel stresses just outside the splice region but within the constant moment region. However, the values of deflection at mid span and at the location of the concentrated loads were measured by all the authors. Due to the sensitivity of the bond strength to the thickness of the concrete covers, especially the
thickness of the smallest cover, the need to have accurate values for the concrete cover thickness was relevant. For this reason, most of the above mentioned authors, accurately measured the clear concrete covers thicknesses, at the splice region, and the spliced bars clear spacing after testing the beams. Beside those researchers, who focused their research on high strength concrete, the nominal concrete compressive strength selected was in the range of 3500 psi to 6000 psi.

The Michigan Technological University published a report in July 2002 discussing the bond properties of MMFX rebars based on the results of an experimental program conducted in their lab. The program included testing one hundred and thirty beam end bond specimens configured in consistence with the ASTM A944 standards. Two bar sizes were selected #4 and #6 and two types of rebars were used; MMFX rebars and conventional carbon Gr. 60 steel rebars for comparison purposes. The bonded lengths ranged between 4-in. and 10-in. for the #4 bars, and 5.5-in. to 12-in. for the #6 bars. Concrete clear cover for all beams was 1½-in. No transverse reinforcement was used along the bonded length in all the specimens. A statistical comparison of MMFX reinforcement test results to predicted values for bond strength of A615 reinforcement was conducted. The comparison revealed that there is no reason to believe the bond strength of MMFX reinforcing bars was less than predicted. The conservatism of the OJB development length relationship, as well as other comparative relationships including AASHTO 16th ed. Standard Specifications and ACI 318-99, predicted lower bond strengths than observed at all bonded lengths. The researchers performed tailed t-tests with a significance level, α, equal to 0.05 to statistically compare the predicted bond strength (OJB equation) with the normalized, with respect to concrete
strength, MMFX experimental bond strength for #4 bars. It was concluded that there is no statistical reason to think that bond strength for #4 MMFX bars is less than the predicted bond strength. Also the T-tests were conducted to compare the normalized MMFX experimental bond strength to normalized A615 experimental bond strength. A conclusion was made that there is no statistical reason to believe that the bond strength for #4 MMFX bars is less than the bond strength of #4 A615 Gr. 60 bars. The same above conclusions were made for #6 MMFX rebars. It was finally reported that no development length reduction factor would be needed for MMFX bars on a one-to-one replacement for A615 steel for #4 and #6 rebars.

In the report that was prepared by the FDOT, two large scale beams one was reinforced with 2 # 6 MMFX, and the other reinforced with 2 # 6 Grade 60 rebars were tested. In both beams a 10 in. lap splice was provided in the mid span. The beams cross sectional dimensions were the same, 12 in. x 18 in., with a 14 ft simply supported span and 2 # 4 stirrups were used to confine the splice zone. Both beams failed in bond failure mechanism before reaching the yield strength of the rebars. From the load deflection curve it was concluded that the bond behavior of the MMFX rebars is equal to or slightly better than that of grade 60 rebars.

Again, like the above mentioned test, but this time using 30.5 in. splice lengths, another two beams were tested by FDOT. It was observed that this splice length was adequate to provide the grade 60 rebars with yield strength. And the same conclusion was made that bond behavior of the MMFX reinforcement is equal or slightly higher than that of Grade 60.
2.9 Descriptive Expressions for Bond

A number of expressions that predicts the bond strength based on the regression analysis of experimental result are presented. All the equations are based on bottom-cast bars results.

2.9.1 Orangun, Jirsa, and Breen (OJB) [1975, 1977]:

They used the statistical analysis technique, to develop their equations. They developed two expressions; one to describe the bond strength of bars with confining transverse reinforcement and the other expression to describe the bond strength for those beams without transverse reinforcement. The database used included 62 beams, 57 of them were bottom cast bars, and five were either side or top cast bars.

According to OJB [1977], for bars not confined with transverse reinforcement, the total bond force is given by:

\[
\frac{L_d}{d_b} = \frac{f_y}{\sqrt{f'_{c}}} - 200 \quad \frac{1}{12} \left[ (C_{min}^' + 0.4d_b) \right] \quad \text{Equation (2.5)}
\]

However, for bars confined with transverse reinforcement the total bond force is given by:

\[
\frac{L_d}{d_b} = \frac{f_y}{\sqrt{f'_{c}}} - 200 \quad \frac{1}{12} \left[ (C_{min}^' + 0.4d_b) + \frac{A_{tr}f_{yt}}{1500 sn} \right] \quad \text{Equation (2.6)}
\]
Where, see figure (2.9);

\( T_b \): the total bond force (lb.)

\( f_{c'} \): concrete compressive strength measured using 6 x 12 in cylinders. (psi)

\( L_d \): development/splice length. (in.)

\( C'_{\text{min}} \): minimum of concrete covers surrounding the bar or half the clear spacing between bars, minimum of \( C_s i \) and (\( C_b \) or \( C_{so} \)) (in.)

\( d_b \): bar diameter. (in.)

\( A_{tr} \): area of each stirrup or tie crossing the potential plane of splitting. (in.\(^2\))

\( s \): spacing of transverse reinforcement. (in.)

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

\( f_y \): specified yield strength of the tested bar. (psi)

\( f_{yt} \): yield strength of the transverse reinforcement. (psi)

Equations (2.5) and (2.6) are limited to cases in which splitting failure, rather than pullout failure governs. So the application, of the above two equations, is restrained by the following condition:

\[
\frac{1}{d_b} \left( C'_{\text{min}} + 0.4d_b \right) + \frac{A_{tr}}{1500sn f_{yt}} \leq 2.5
\]

Equation (2.7)

2.9.2 *Darwin et al. [1992]:*

By reanalyzing the data used by Orangun et al. [1975, 1977] and including the effect of the relative value of \( C'_{\text{max}} \) and \( C'_{\text{min}} \), Darwin et al. [1992] establish their expression for
bars not confined with transverse reinforcement as given below:

\[
\frac{L}{d_b} = \frac{f_y}{\sqrt{f'_{c}}} - 300 \frac{0.1177 \left( C'_{\text{min}} + 0.5 d_b \right)}{d_b} \left[ 0.08 \frac{C'_{\text{max}}}{C'_{\text{min}}} + 0.92 \right]
\]

Equation (2.8)

Where, see figure (2.9);

\( C'_{\text{min}} \): minimum of concrete covers (bottom and side) surrounding the bar and half the clear spacing between bars, minimum of \( (C_b \text{ or } C_{so}) \) and \( C_{si} \) (in).

\( C'_{\text{max}} \): maximum of concrete covers (bottom and side) surrounding the bar and half the clear spacing between bars, minimum of \( (C_b \text{ or } C_{so}) \) and \( C_{si} \) (in).

2.9.3 Darwin et al. [1996]:

They used a large database including 133 splice and development length specimens with bars not confined by transverse reinforcement and 166 specimens with bars confined by transverse reinforcement. All the specimens were bottom cast bars. They concluded that \( f'_c^{1/4} \) provides a more accurate representation of the concrete strength on development and splice strength than the currently used \( f'_c^{1/2} \). They also integrated the effect of the relative rib area \( R_r \) in their expression because they observed that this factor has a significant effect on the bond strength of bars confined by transverse reinforcement. Based on their analytical study the best fit equation that can predict the development length for bars without transverse reinforcement confinement is:
And, for bars confined by transverse reinforcement, the following equation was obtained:

\[
\frac{L_s}{d_b} = \frac{\frac{f_y}{f_c^{1/4}} - 2130 \left( 0.1 \frac{C_{\max}}{C_{\min}} + 0.9 \right)}{80 \cdot 0.21 \left( C_{\min} + 0.5d_b \right) \left[ 0.1 \frac{C_{\max}}{C_{\min}} + 0.92 \right]} \quad \text{Equation (2.9)}
\]

\[
\frac{L_s}{d_b} = \frac{\frac{f_y}{f_c^{1/4}} - 2130 \left( 0.1 \frac{C_{\max}}{C_{\min}} + 0.9 \right) - \frac{66}{A_b}}{80 \cdot 0.21 \left( C_{\min} + 0.5d_b \right) \left[ 0.1 \frac{C_{\max}}{C_{\min}} + 0.92 \right] + \frac{35 \cdot 33}{sn} t_r t_d A_{tu}} \quad \text{Equation (2.10)}
\]

Where;

- \( C_{\max} \): maximum of \( (C_b, C_s) \)
- \( C_b \): clear bottom cover.
- \( C_s \): minimum of \( (C_{so}, C_{si} + 0.25 \text{ in}) \)
- \( C_{so} \): clear side cover.
- \( C_{si} \): half the clear spacing between bars.
- \( C_{\min} \): the minimum of \( (C_b, C_s) \)

\( t_r = 9.6 \ R_r + 0.28 < 1.72 \) and \( t_d = 0.72 \ d_b + 0.28 \)

\( N \): total number of stirrups within the development length.

\( R_r \): Relative rib area, is the ratio of the bearing area (projected rib area on a plane normal to the bar axis) to the shearing area (the surface area of the bar between 2 adjacent ribs), the
shaded area in Figure (2.9b)

Like all the above mentioned expressions and to ensure that the splitting mode of failure will govern, the above equation is applicable only if the following condition is met:

\[
\frac{1}{d_b} \left( C_{\text{min}} + 0.5d_b \right) \left( 0.1 \frac{C_{\text{max}}}{C_{\text{min}}} + 0.9 \right) + \left( 35.3 t_d \frac{A_p}{SN} \right) \leq 4.0 \quad \text{Equation (2.11)}
\]

2.9.4 Zuo and Darwin [1998, 2000]:

They expanded the work of Darwin et al. [1996b] by increasing the data base specially specimens made with high strength concrete (f_{c'} > 8000 psi). The data base they used reached 171 specimens containing bars confined by transverse reinforcement and 196 specimens containing bars confined by transverse reinforcement. All bars were bottom cast. Their results supported the early observation made by Darwin et al. that \( f_{c'}^{1/4} \) accurately represent the contribution of concrete compressive strength to bond strength for bars not confined by transverse reinforcement. They also observed that \( f_{c'}^p \) with \( p \) between \( \frac{3}{4} \) and 1.0 best represent the effect of concrete strength on the contribution of confining of transverse reinforcement to bond strength. They selected \( p = 3/4 \) for their equation. For bars not confined by transverse reinforcement, the descriptive equation is:

\[
\frac{L_d}{d_b} = \frac{\frac{f_y}{f_{c'}^{1/4}} - 2350 \left( 0.1 \frac{C_{\text{max}}}{C_{\text{min}}} + 0.9 \right)}{76.3 \left( C_{\text{min}} + 0.5d_b \right) \left( 0.1 \frac{C_{\text{max}}}{C_{\text{min}}} + 0.92 \right) \left( 0.92 \right)} \quad \text{Equation (2.12)}
\]
However, for bars confined by transverse reinforcement, the descriptive equation is

\[
\frac{L_{c}}{d_{b}} = \frac{f_{y}}{f_{c}^{1/4}} - 2350 \left( 0.1 \frac{C_{\text{max}}}{C_{\text{min}}} + 0.9 \right) \frac{76.3}{d_{b}} \left( C_{\text{min}} + 0.5 d_{b} \right) \left( 0.1 \frac{C_{\text{max}}}{C_{\text{min}}} + 0.92 \right) + \left( \frac{0.52 t_{r} t_{d} A_{tr}}{sn} \right) f_{c}^{1/2} \text{ Equation (2.13)}
\]

Where;

\[ t_{r} = 9.6 R_{r} + 0.28 < 1.72 \text{ and } t_{d} = 0.78 d_{b} + 0.22 \]

To limit the applicability of the above two equations to cases in which a splitting failure governs:

\[
\frac{1}{d_{b}} \left( C_{\text{min}} + 0.5d_{b} \right) \left( 0.1 \frac{C_{\text{max}}}{C_{\text{min}}} + 0.9 \right) + \left( 0.52 t_{r} t_{d} \frac{A_{tr}}{sn} \right) f_{c}^{1/2} \leq 4.0 \text{ Equation (2.14)}
\]

2.9.5 **ACI Committee 408 [2001]:**

Using the ACI committee 408 database 10-2000 adopted and updated the expressions developed by Zuo and Darwin [1998, 2000], equation (2.12) and (2.13), with minor numerical changes and rounding figures, to become the following two equations:
Where

\( t_r = 9.6 \, R_r + 0.28 < 1.72 \) and \( t_d = 0.78 \, d_b + 0.22 \)

The same restriction that applies to Zuo and Darwin [2000] expression applies to the ACI 408 committee equation:

\[
\frac{1}{d_b} \left( (C_{\min} + 0.5 d_b) \left( 0.1 \frac{C_{\max}}{C_{\min}} + 0.9 \right) + \left( \frac{0.52 t_r t_d A_{st}}{sn} \right) f'^{1/4} c \right) \leq 4.0 \quad \text{Equation (2.17)}
\]
2.10 Design Expressions for Bond

2.10.1 Design Expression in chapter 12 of ACI 318-02:

ACI code adopted the OJB equations for both cases of with and without transverse reinforcement confinement. The code used the same expressions for $L_d/d_b$ but substituted $f_y$ for $f_s$ and incorporated four parameters namely; $\alpha$, $\beta$, $\lambda$, and $\gamma$ to account for the effect of bar location, epoxy coating, lightweight aggregate concrete, and reinforcement size factor respectively. They also neglected the 200 number in the numerator of the OJB equation and multiplied the 1/12 constant by 0.9 to obtain 3/40 constant, and introduced the following expression:

$$\frac{L_d}{d_b} = \frac{3f_y\alpha\beta\lambda\gamma}{40\sqrt{f_{c'}^2/c + K_w}}$$

Equation (2.18)

Where:


d_b: Bar Diameter, (in).

$f_y$: Bar yield Strength, (psi).

$\alpha$: reinforcement location factor; (top cast or bottom cast)

$\beta$: Coating factor (coated or uncoated bars)

$\lambda$: Light weight aggregate factor

$\gamma$: Reinforcement size factor ( =0.8 for # 6 bars and smaller, and = 1 for otherwise)

$f_{c'}$: Concrete compressive strength measured using 6 x 12 in cylinders, (psi).
\[ K_n = \frac{A_{tr} f_{yt}}{1500sn} \]

\( C'_{\text{min}} \): minimum of concrete covers (bottom and side) surrounding the bar and half the clear spacing between bars, minimum of \( (C_b \text{ or } C_{so}) \) and \( C_{si} \), (in.).

\( A_{tr} \): area of each stirrup or tie crossing the potential plane of splitting, (in²).

\( f_{yt} \): yield strength of the transverse reinforcement, (psi).

\( s \): spacing of transverse reinforcement, (in).

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

\[ c = C'_{\text{min}} + 0.5d_b \]

To reduce the probability of pullout failure, the following requirement was imposed by the ACI-318-02 code:

\[ \frac{1}{d_b} \left[ (C'_{\text{min}} + 0.5d_b) + \frac{A_{tr}}{1500sn} f_{yt} \right] \leq 2.5 \quad \text{Equation (2.19)} \]
2.10.2 ACI Committee 408 Design Expressions:

Converting an empirical, best fit equation to design expression involves the incorporation of a strength reduction factor, $\Phi$, to ensure a realistic low probability of failure. The $\Phi$ factor for bond depends on the $\Phi$ factor for tensions and the load factors for dead and live loads. Based on a Monte Carlo analysis, $\Phi$ for bond is 0.82, in case the tension $\Phi$ factor is 0.90, the dead load factor is 1.2, and the live load factor is 1.6. For a tension $\Phi$ factor of 0.90, a dead load factor is 1.4, and a live load factor is 1.7 the $\Phi$ factor for bond will be 0.92. Based on that, the ACI committee 408 developed two expressions; one with the bond $\Phi$ factor of 0.82 and the other with $\Phi$ factor equals 0.92.

2.10.2.1 ACI Committee 408 Design Expressions using $\Phi$ factor = 0.92:

The 0.82 $\Phi$ factor was multiplied by the $f_{c'}$ term in equation (2.6), and the effect of the bar location during casting, presence of epoxy coating, and lightweight aggregate are considered in the design expression through multiplying the equation by $\alpha$, $\beta$, and $\lambda$. After rounding the numbers the following formula was obtained:

$$
\frac{L_\text{d}}{d_b} = \frac{f_y}{f_{c'}^{1/4}} - 2200 \left( 0.1 \frac{C_{\text{max}}}{C_{\text{min}}} + 0.9 \right) \frac{70 \left( C_{\text{min}} + 0.5 d_b \right) \left[ 0.1 \frac{C_{\text{max}}}{C_{\text{min}}} + 0.92 \right] + 0.52 \frac{t_r t_d A_p}{sn}}{d_b}
$$

Equation (2.20)
2.10.2.2 ACI Committee 408 Design Expressions using $\Phi$ factor = 0.82:

The 0.82 $\Phi$ factor was multiplied by the $f_{c'}$ term in equation (2.6), and the effect of the bar location during casting, presence of epoxy coating, and lightweight aggregate are considered in the design expression through multiplying the equation by $\alpha$, $\beta$, and $\lambda$. After rounding the numbers the following formula was obtained:

$$
\frac{L_d}{d_b} = \frac{f_y}{f_{c'}^{1/4}} - 1970 \left( 0.1 \frac{C_{\text{max}}}{C_{\text{min}}} + 0.9 \right)
$$

$$
\frac{62 \left( C_{\text{min}} + 0.5 d_b \right) \left[ 0.1 \frac{C_{\text{max}}}{C_{\text{min}}} + 0.92 \right] + 0.52 \frac{t_r t_d A_{yr}}{sn}}{d_b}
$$

Equation (2.21)

A comparison was conducted by the ACI committee 408, and reported in the ACI committee 408 report in October 2001, between the predicted values for bond capacity ($T_b$) from the five descriptive equations for bars not confined by transverse reinforcement, [Equations (2.5), (2.8), (2.9), (2.12), and (2.15)] and the test results from ACI 408 database 10-2001. This database included 635 development and splice tests of uncoated reinforcing bars, and was limited to normal weight concrete specimens. The concrete compressive strengths for the test results included in the data base were obtained by testing concrete cylinders rather than cubes. The tests included in the data base could be classified according to the bar placement, 478 tests for bottom-cast, 111 tests for top-cast, and 46 tests for side-
cast. The results of this comparison are shown in table (2.1). Table (2.1) shows that for bars not confined by transverse reinforcement the average test/predicted ratio are within 3 percent. OJB equation shows the highest extremes between maximum and minimum test/predicted values, while the ACI 408 equation shows the smallest difference between them. The minimum scatter is exhibited by the expression obtained by Zuo and Darwin [2000] and ACI committee 408, as they give a coefficient of variation of 0.111. On the other hand, the highest value for the coefficient of variation, 0.202, was obtained from the test/predicted ratio comparisons made with the OJB equations. Figure (2.10) shows the relationship between the test/predict ratio and the concrete compressive strength for the five different descriptive equations. It is clear that OJB [1977] and Darwin et al. [1992] equations, which consider the effect of concrete strength as $\sqrt{f'_c}$, exhibit a remarkable decrease in test/predict ratio as the compressive strength increases. The reason for this trend is that these two equations were formulated based on test results that included a narrow range of compressive strengths. On the other hand, the other three expressions, which considered the impact of concrete strength as $f'_c$, demonstrate significantly less variation in the test/predict ratio, with a very slight increase in the ratio as the concrete strength increases from 3000 to 16000 psi.

Another comparison was made, but this time for bars confined by transverse reinforcement using the four equations, [(2.6), (2.10), (2.13), and (2.16)], and the results are provided in table (2.2). The average test/predicted ratios ranged from 1.074 for OJB [1977] to a low of 0.989 for Zuo and Darwin [2000]. The best overall match was obtained by ACI 408 expression with an average test/predict ratio of 1.00 and a coefficient of variation of 0.12. In figure (2.11), it is obvious that the test/predict ratio for the equations developed by
Zuo and Darwin [2000] and ACI 408 committee are remarkably insensitive to the variation compressive strength. Because these two expressions consider the contribution of the concrete as a function of $f_c^{3/4}$ and the contribution of the transverse reinforcement as function of $f_c^{3/4}$. However for the other two expressions, the contribution of the concrete and the contribution of the transverse reinforcement are treated differently. In OJB [1977] equation, the contribution for both concrete and transverse reinforcement is considered to be influenced by the concrete compressive strength as function of $\sqrt{f_c'}$, while in Darwin et al. [1996] this contribution, for both concrete and transverse reinforcement, is considered to be function in $f_c^{'}$. That is why the OJB equation shows a high test/predict ratios for low strength concrete, and low test/predicted ratios for higher strength concrete as shown in figure (2.11). In the contrast Darwin et al. [1996] expression, provides low ratios for lower strength concrete and high test/predict ratios as the strength increases.

Tables (2.3) and (2.4) provides another comparison done by the ACI committee 408 between the test/predict ratios for the different design expressions shown in equations, [(2.18), (2.20), and (2.21)]. The comparison reflects that the test/predict ratios for the ACI 318-02 equation shows higher scatter than those for ACI 408. The test/predict ratio ranged from 0.704 to 2.192 for the bars confined with transverse reinforcement. Overall, the ACI 318-02 provides a higher average test/predict ratios and higher coefficients of variation than do the equations of the ACI committee 408. That implies that the ACI 318-02 requires longer development length than ACI 408. The ACI 408 equation, equation (2.21), that was based on load factors and strength reduction factors presented in chapter 9 of ACI 318-02 provides average test/predict ratios that are slightly higher than those given by the ACI 318-02.
2.11 MFX Properties:

The fundamental mechanical properties of the MMFX steel rebars and the behavior of the MMFX rebars in composite action with concrete in concrete structures were studied, [NCSU-CFL Report No. 02-04]. The testing focused on the mechanical properties in tension and in compression, shear strength, fatigue strength, effect of bend on tensile strength of bent rebar (stirrup), bond strength and development length, and the behavior of MMFX rebars as compression steel in reinforced concrete columns.

2.11.1 MMFX Tension Properties:

The tension test was conducted for three different sizes of MMFX steel rebar (#4, #6, and #8) using five straight specimens for each rebar size. The major objective was to determine the characteristics of the MMFX steel rebars in tension. The experimental results included evaluating the modulus of elasticity ($E$), ultimate stress ($f_{u}$) and corresponding strain, and ultimate strain ($\varepsilon_{u}$) of the material in tension. Summary of significant test results is given in table (2.5) for the #4, #6 and #8 rebars, respectively. The engineering stress-strain curves for each rebar size #4, #6, and #8 are shown in figure (2.12), (2.13), and (2.14) respectively.

2.11.2 MMFX Composite Behavior with Concrete:

As part of the test program conducted to determine the performance of the MMFX rebars as reinforcing bars embedded in concrete members; a bond investigation was conducted [NCSU-CFL Report No. 02-04]. The objective was to determine the bond strength
and the development length for three different rebar sizes (#4, #6, and #8) under certain levels of transverse confinement. Three reinforced concrete specimens of T-section configuration were used. Each specimen contained six MMFX rebars of the same size. Different embedment lengths of about $8db$, $12db$, $38db$, $58db$, $78db$ and $98db$ were examined for each selected MMFX rebar size, where $db$ is the diameter of the rebar. The concrete specimens were heavily reinforced with A615 Grade 60 ordinary steel to avoid premature failure along the de-bonded lengths of the MMFX rebars. A schematic showing a typical configuration for one of the specimens together with test setup used is shown in figure (2.15). The results of this bond study are summarized in table (2.6).

### 2.12 Bar Stresses at Bond Failure

To determine the bar stress at bond failure, both the bending moment, corresponding to this failure, and the properties of the non-linear inelastic materials involved should be known. In general there are three approaches that have been used to calculate the force in the rebars at bond failure; the moment curvature method, the working stress method, and ultimate strength method. In the moment curvature method, stress-strain relationships are assumed for both the concrete and the steel. Zuo [1998] performed an analysis to compare the bar stresses using three methods. He concluded that for beams, in which the bars did not yield, the working stress method overestimates bar stresses for high strength concrete and underestimates bar stresses for normal strength concrete, compared to the moment-curvature method. However, the ultimate strength method underestimates the bar stresses for both high and low strength concrete. According to these results, the moment curvature method was one
used to compute the bar stresses at bond failure. The following equation represents the Hognestad [1951] parabolic equation that was adopted, in this research program, to predict the concrete stress strain diagram:

\[
f_c = 0.85 f'_c \left( \frac{2 \varepsilon_c}{\varepsilon_o} - \left( \frac{\varepsilon_c}{\varepsilon_o} \right)^2 \right) \quad \text{Equation (2.22)}
\]

Where: \( \varepsilon_c \) is the concrete strain corresponding to the compressive stress \( f_c \), and \( \varepsilon_o \) is the concrete strain corresponding to the maximum concrete stress. The value of \( \varepsilon_o \) is given by:

\[
\varepsilon_c = \frac{1.7 f'_c}{E_c} \quad \text{Equation (2.23)}
\]

Where: \( E_c \) is the elastic modulus of concrete and is based on the expression in section 8.5 of the ACI 318-02 code and is given by:

\[
E_c = 57,000 \sqrt{f'_c} \quad \text{Equation (2.24)}
\]

The stress-strain curves for the MMFX steel that was used in the moment curvature analysis were the average stress-strain curves obtained from the mechanical properties analysis for different bar sizes given in figures (2.12), (2.13), and (2.14).
Tension force distribution between the two cracks.

Bond force distribution between the two cracks.

Constant bending moment region

Tension force in steel

Bond stresses in steel

Bond force distribution between the two cracks.

Figure (2.1) Tension force and Bond force distribution along the bonded length between two cracks
Figure (2.2) Tension force and Bond force distribution along the rebar in the shear span
Figure (2.3) Bond Force develops in Flexural Members due to Moment

Figure (2.4) Force Transfer Mechanisms through Bond in Deformed Bars
Figure (2.6) Formation of Splitting Cracks in the hoop direction around the bar

Figure (2.5) Splitting Cracks running through Concrete Covers and between bars

Cracks Opening as a result of tensile radial stresses

Radial tensile stresses as a result of wedging action of the bar ribs
Figure (2.7) Pullout Failure associated with Shear Cracks

Figure (2.8) Types of Bond Specimens
Figure (2.9a) Schematic describing the Physical Meaning of the Notations

Figure (2.9b) Cross Section in the Bar
Figure (2.10) Test/predict Ratio vs. Concrete Compressive Strength for Bars Not Confined with Transverse Reinforcement
Figure (2.11) Test/Predict Ratio vs. Concrete Compressive Strength for Bars Confined with Transverse Reinforcement
Figure (2.12) Stress Strain Curve for # 4 MMFX Rebar
Figure (2.13) Stress Strain Curve for #6 MMFX Rebar
Figure (2.14) Stress Strain Curve for #8 MMFX Rebar

E=29,000ksi

Extensometer removed
Figure (2.15) Elevation and plan view for the tested specimen and the test setup
Table (2.1) test/predicted ratios for bars not confined by transverse reinforcement using descriptive equations.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum</td>
<td>1.555</td>
<td>1.383</td>
<td>1.342</td>
<td>1.304</td>
<td>1.288</td>
</tr>
<tr>
<td>Minimum</td>
<td>0.505</td>
<td>0.528</td>
<td>0.719</td>
<td>0.729</td>
<td>0.724</td>
</tr>
<tr>
<td>Average</td>
<td>1.030</td>
<td>1.014</td>
<td>1.020</td>
<td>1.010</td>
<td>1.000</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>0.208</td>
<td>0.189</td>
<td>0.118</td>
<td>0.113</td>
<td>0.111</td>
</tr>
<tr>
<td>COV**</td>
<td>0.202</td>
<td>0.187</td>
<td>0.116</td>
<td>0.111</td>
<td>0.111</td>
</tr>
</tbody>
</table>

* Orangun, Jersa and Breen [1975, 1977]
** Coefficient of Variation

Table (2.2) test/predicted ratios for bars confined by transverse reinforcement using descriptive equations.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum</td>
<td>1.902</td>
<td>1.479</td>
<td>1.309</td>
<td>1.333</td>
</tr>
<tr>
<td>Minimum</td>
<td>0.595</td>
<td>0.776</td>
<td>0.739</td>
<td>0.755</td>
</tr>
<tr>
<td>Average</td>
<td>1.074</td>
<td>1.052</td>
<td>0.989</td>
<td>1.002</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>0.255</td>
<td>0.132</td>
<td>0.119</td>
<td>0.121</td>
</tr>
<tr>
<td>COV**</td>
<td>0.238</td>
<td>0.125</td>
<td>0.121</td>
<td>0.120</td>
</tr>
</tbody>
</table>

* Orangun, Jersa and Breen [1975, 1977]
** Coefficient of Variation
Table (2.3) test/predict ratio for bars not confined by transverse reinforcement using design expressions.

<table>
<thead>
<tr>
<th>For $f'_{c}$&lt; 1000psi</th>
<th>ACI 318-02</th>
<th>ACI 408 (eq. 2.20)</th>
<th>ACI 408 (eq. 2.21)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum</td>
<td>1.908</td>
<td>1.401</td>
<td>1.574</td>
</tr>
<tr>
<td>Minimum</td>
<td>0.756</td>
<td>0.749</td>
<td>0.843</td>
</tr>
<tr>
<td>Average</td>
<td>1.229</td>
<td>1.089</td>
<td>1.224</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>0.229</td>
<td>0.108</td>
<td>0.121</td>
</tr>
<tr>
<td>COV**</td>
<td>0.186</td>
<td>0.099</td>
<td>0.099</td>
</tr>
</tbody>
</table>

Table (2.4) test/predict ratio for bars confined by transverse reinforcement using design expressions.

<table>
<thead>
<tr>
<th>For $f'_{c}$&lt; 1000psi</th>
<th>ACI 318-02</th>
<th>ACI 408 (eq. 2.20)</th>
<th>ACI 408 (eq. 2.21)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum</td>
<td>2.192</td>
<td>1.525</td>
<td>1.715</td>
</tr>
<tr>
<td>Minimum</td>
<td>0.704</td>
<td>0.845</td>
<td>0.951</td>
</tr>
<tr>
<td>Average</td>
<td>1.237</td>
<td>1.15</td>
<td>1.294</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>0.311</td>
<td>0.155</td>
<td>0.175</td>
</tr>
<tr>
<td>COV**</td>
<td>0.251</td>
<td>0.135</td>
<td>0.135</td>
</tr>
<tr>
<td>Bar Size</td>
<td>Proportional Limit Strength (ksi)</td>
<td>Young's Modulus of Elasticity (ksi)</td>
<td>Yield Strength based on 0.2% offset (ksi)</td>
</tr>
<tr>
<td>----------</td>
<td>----------------------------------</td>
<td>-------------------------------------</td>
<td>------------------------------------------</td>
</tr>
<tr>
<td># 4</td>
<td>91.4</td>
<td>29,000</td>
<td>116</td>
</tr>
<tr>
<td># 6</td>
<td>83.5</td>
<td>29,000</td>
<td>119.91</td>
</tr>
<tr>
<td># 8</td>
<td>88.24</td>
<td>29,000</td>
<td>118.35</td>
</tr>
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Table (2.5) results of the tension tests for # 4, # 6, and # 8 MMFX rebars.
Table (2.6) summary of the bond study results

<table>
<thead>
<tr>
<th>Embedment Length</th>
<th>4in</th>
<th>9in</th>
<th>19in</th>
<th>29.25in</th>
<th>39in</th>
<th>49in</th>
</tr>
</thead>
<tbody>
<tr>
<td>8d</td>
<td>13830</td>
<td>31940</td>
<td>34820</td>
<td>32725</td>
<td>33526</td>
<td>33418</td>
</tr>
<tr>
<td>18d</td>
<td>2.2</td>
<td>2.26</td>
<td>1.16</td>
<td>0.71</td>
<td>0.55</td>
<td>0.43</td>
</tr>
<tr>
<td>Axial Stress (ksi)</td>
<td>70.6</td>
<td>163</td>
<td>178</td>
<td>167</td>
<td>171</td>
<td>170.5</td>
</tr>
<tr>
<td>Slip at Free End (in)</td>
<td>0.019864</td>
<td>0.038281</td>
<td>0.003812</td>
<td>0.000342</td>
<td>0.000150</td>
<td>0.000014</td>
</tr>
<tr>
<td>Failure Mode</td>
<td>Slip</td>
<td>Slip</td>
<td>Rupture</td>
<td>Rupture</td>
<td>Rupture</td>
<td>Rupture</td>
</tr>
</tbody>
</table>

Table 7: Results of bond test of #6MMFX rebars

<table>
<thead>
<tr>
<th>Embedment Length</th>
<th>7in</th>
<th>14in</th>
<th>28in</th>
<th>44in</th>
<th>59in</th>
<th>74in</th>
</tr>
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<tbody>
<tr>
<td>9.3d</td>
<td>36875</td>
<td>71626</td>
<td>74084</td>
<td>75572</td>
<td>65951</td>
<td>---</td>
</tr>
<tr>
<td>Max. Bond Stress (ksi)</td>
<td>2.24</td>
<td>2.10</td>
<td>1.08</td>
<td>0.73</td>
<td>0.47</td>
<td>---</td>
</tr>
<tr>
<td>Axial Stress (ksi)</td>
<td>84</td>
<td>163</td>
<td>168</td>
<td>172</td>
<td>150</td>
<td>---</td>
</tr>
<tr>
<td>Slip at Free End (in)</td>
<td>0.04975</td>
<td>0.04891</td>
<td>0.02927</td>
<td>0.01247</td>
<td>0.00156</td>
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</tr>
<tr>
<td>Failure Mode</td>
<td>Slip</td>
<td>Slip</td>
<td>Rupture</td>
<td>Rupture</td>
<td>Anchorage (Rebar didn't fail)</td>
<td>not tested</td>
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</table>

Table 8: Results of bond test of #8MMFX rebars

<table>
<thead>
<tr>
<th>Embedment Length</th>
<th>8in</th>
<th>18.5in</th>
<th>38.25in</th>
<th>57.5in</th>
<th>78in</th>
<th>98.25in</th>
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</thead>
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<tr>
<td>8d</td>
<td>48095</td>
<td>127739</td>
<td>131886</td>
<td>140736</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Max. Bond Stress (ksi)</td>
<td>1.91</td>
<td>2.2</td>
<td>1.1</td>
<td>0.78</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Axial Stress (ksi)</td>
<td>61</td>
<td>162</td>
<td>167</td>
<td>178</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Slip at Free End (in)</td>
<td>0.043852</td>
<td>0.100137</td>
<td>0.131684</td>
<td>0.041556</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Failure Mode</td>
<td>Slip</td>
<td>Slip</td>
<td>Anchorage (Rebar didn't fail)</td>
<td>Anchorage (Rebar didn't fail)</td>
<td>not tested</td>
<td>not tested</td>
</tr>
</tbody>
</table>
3.1 Introduction

Two test programs were conducted to examine the bond characteristics and the development length of the MMFX rebars as flexural reinforcement for concrete beams. The objective of the first program was to obtain preliminary results for the bond strength of the MMFX with concrete. The second program represents a more in depth investigation about the bond properties and development length. Checking the applicability of using the current code design expression for development length and other equations to predict the MMFX development length was also an additional objective.

3.2 Test Program I: Beam End Specimens

3.2.1 General

This Chapter describes the first experimental program, including a detailed description for the tested specimens, mechanical properties of the materials used, the test setup configuration and the instrumentation used. The objectives of this experimental work were to examine bond behavior of the MMFX steel rebars with concrete in flexural members and to evaluate the development length. The key parameters considered were the bar size, the bonded length, amount of transverse reinforcement, and bar location. The test program included preparing and testing four half concrete beams each reinforced with one MMFX bar. The specimens were tested as cantilever beams by applying a tension force to the MMFX bar and supporting the beam at three different locations creating a cantilever effect.
All the beams had the same concrete dimensions with a rectangular cross.

3.2.2 Test Specimens

The four beam end specimens included in this experimental program were all cast on the same time. The dimensions of all the specimens were 14 inches in width, 20 inches in height and 80 inches long. Three specimens were each reinforced with one #4 MMFX rebar in tension and the fourth specimen was reinforced with one #8 rebar. Three specimens have the same amount of transverse reinforcements, #3 @ 9 in., along the bonded length, while the fourth specimen had three times this amount. Three specimens were cast with the MMFX bar in the bottom while the fourth was cast with the MMFX bar at top. The bonded length was 14 in. constant for three specimens and 34 in. for the fourth. To avoid conical failure, the first four inches of the MMFX bar from the concrete surface were de-bonded using PVC plastic tubes. The same tubes were used to de-bond the rest of the bar length. In all the specimens, the MMFX bar was extended one inch from the end of the specimen. All the specimens were reinforced with two #6 grade 60 steel bars in the compression zone to keep the spacing between the stirrups during casting. Four #8 grade 60 steel rebars were placed along the sides of the stirrups in the tension side of the beam to carry tension force along the de-bonded length of the specimen. The clear bottom cover in all the specimens was 1 ½ in. The nominal concrete dimensions, concrete covers thicknesses, bonded lengths, amount of transverse reinforcement for all the beams are given in table (3.1). A schematic presenting typical beam configuration in elevation, plan and cross section is shown in figure (3.1). The reinforcement details are presented in figure (3.2).
3.2.3 **Materials Properties**

3.2.3.1 **Reinforcing Steel:**

All the reinforcing bars that were tested were MMFX micro-composite steel. However all the steel used for the stirrups and compression steel were rolled deformed steel bars grade 60 steel satisfying ASTM A615 requirements. The average relative rib area for # 4 bars was found to be 0.1119 and for # 8 bars 0.09. To fully characterize the steel used in the tests, the mechanical and geometrical properties of the tested MMFX bars are reported in table (3.2).

3.2.3.2 **Concrete:**

Concrete was supplied by a local ready-mix plant. All the specimens were cast at one time from the same batch of concrete to ensure consistency in the concrete properties. The target concrete compressive strength was 6000 psi while the average compressive strength as measured, using 4 in. x 8 in. cylinders, on the day of testing the specimens ranged between 5230 psi and 5320 psi as given in table (3.3). Type I Portland cement, river sand and # 67 coarse aggregate stones were used in preparing the concrete mix.

3.2.4 **Test Setup**

All the beams were tested as cantilever beams by applying a tension force in the MMFX bar. The distance between the compression zone on the specimen surface and the MMFX bar was selected following the ASTM A944-99 standard. Two concrete pedestals were used to support the specimen’s self weight. A hydraulic jack together with a wedge and
a pulling head were used to apply a tension force to the MMFX bar. A 200 kips capacity load cell was used for measuring the exerted force. The hydraulic jack acted against a steel box section that was bolted to an HSS steel beam that in turn bear on the specimen surface through a steel plate. To support the beam against rotation, an HSS steel box section beam was placed on the top of the beam and was hooked to the laboratory floor through two dewed-age bars. Figures (3.3a) and (3.3b) present a schematic showing the elevation and plan view for the test setup.

3.2.5 Instrumentations

Electric resistance strain gages, LVDTs and extensometers were all the instrumentations used in testing all the beams. All LVDTs and extensometers were calibrated before starting the test. In general the Constructed Facilities Laboratory (CFL), which belongs to NCSU, is accredited by the International Conference of Building Officials (ICBO). In addition the micrometers and the instruments used for calibration are calibrated every year by an accredited laboratory that is traceable to the National Institute of Standards and Technology (NIST). A typical configuration for the instrumentations that have been used for all the beams is defined in the following lines. Three, 6mm (0.24 in) gage length, strain gages were mounted on the external bottom surface of the MMFX rebar (facing the concrete cover). The three strain gages were distributed along the bonded length of the bar at a distance of \( l_b/6 \) and \( l_b/2 \) and \( 5l_b/6 \) from the beginning of the bonded length; where \( l_b \) is the bonded length. To install the strain gages on the surface of the # 4 MMFX rebars, one rib has to be partially grinded, this rib damage was not needed in case of the # 8 rebars. A 2 in. gage
length extensometer was installed on the free loaded end of the MMFX rebar to cross check the bar stresses with that measured by the load cell. Two LVDTs were used to measure the slip of the MMFX rebar at its loaded and unloaded ends. A typical schematic showing the configuration of the different instrumentations used is given in figure (3.4).

3.2.6 Test Procedure

The monotonic load was continuously applied by the hydraulic jack through a stroke control option. The loading rate was in range of 2.5-3.9 kips/min for the specimens reinforced with # 4 bars and 12.6 kips/min for the one reinforced with # 8 bar. The readings of the instrumentations including the load cell were monitored and recorded by means of a MEGADAC data acquisition system at a rate of 1 scan/2 seconds. The test duration varied from one specimen to the other and ranged between 9 and 13 min. except for the 8 bar specimen, where the test lasted for 7.5 min. only due to bond failure achieved.

3.3 Test program II: Splice Specimens

3.3.1 General:

This Chapter describes the experimental program, including detailed description for the tested specimens, mechanical properties of the materials used, the test setup configuration and the instrumentation used. The objectives of this experimental work were to examine bond behavior of the MMFX steel rebars with concrete in flexural members and to evaluate the development length. The key parameters considered were the bar size, and the splice length. The experimental program included preparing and testing eight large scale concrete
beams reinforced with MMFX bars spliced at the mid span. The specimens were tested as simply supported beams under the action of two concentrated loads to subject the beam to constant moment along the spliced length. Four specimens were reinforced with 2 # 6 MMFX bars and the other four were reinforced with 1 # 8 MMFX bar. All the spliced lengths lied within the constant moment region. Four specimens were having a T- shaped section configuration while the other four were having a rectangular cross section to ensure failure at the splice location rather than crushing of the concrete.

3.3.2 Test Specimens

Beam-splice specimens included in this experimental investigation were divided into four groups. Each group contained two specimens. The specimens of each group were identical in their nominal concrete dimensions but different in the number and size of the reinforcing MMFX bars. In the first two groups, the specimens were rectangular in cross section while in the last two groups a T-shaped cross section was selected. The spliced lengths varied from one specimen to the other and ranged from 1 ft to 6 ft. To minimize the confinement effect caused by the applied forces on the spliced length, the distance between the end of the splice length and the center line of the applied load, in all the tested beams, was always more than 1 ft. In the specimens reinforced with # 8 MMFX bars, and in order to provide the required level of confinement around the spliced bars, closed stirrups perpendicular to the beam axis were evenly distributes along the splice length. The flange thickness for all the T-shaped beams was 4 inches and the flange was reinforced with a 4 in. x 4 in. welded wire fabric mesh size # 1. To prevent any premature shear failure, closed
stirrups were spaced every 5 in. along the shear span in all the specimens. All the specimens were reinforced with 2 # 4 steel bars as the top reinforcement to keep the stirrups in their position during casting and to carry tensile stresses that could develop during handling. The nominal bottom and side covers and the spacing between spliced bars were kept constant for all the beams reinforced with # 6 MMFX bars. The selected values were $1.8d_b$, $3d_b$, and $6d_b$ respectively where $d_b$ is the bar diameter. The same consistency applies for beams reinforced with # 8 bars, but with different values of $1.375d_b$ and $5d_b$ respectively. No chairs were used at the location of the splice to support the steel cage. U shaped half length stirrups were used in all the beams reinforced with # 6 bars. These stirrups were distributed along the top reinforcement throughout the constant moment region to resist any possible buckling of the compression reinforcement. Four stirrups were added at each end of the beam, beyond the support section, to compensate for the absence of the bar bends. Variation in the specimens’ dimensions used in this program was selected to achieve different stress levels in the splices at failure. The nominal concrete dimensions, concrete covers thicknesses, splice lengths, reinforcement details for all the beams are given in table (3.4). A schematic presenting typical beam configuration in elevation, plan and cross section is shown in figure (3.5). The reinforcement details are given in figure (3.6). Measured dimensions of the specimens tested in this program are given in table (3.5).

3.3.3 Materials Properties

3.3.3.1 Reinforcing Steel:

All the reinforcing bars that were tested were MMFX micro-composite steel.
However all the steel used for the stirrups and compression steel were rolled deformed steel bars grade 60 steel satisfying ASTM A615 requirements. In the beam designations shown in table (3.4), the first letter of the designation which is “B” stands for Beam; the middle number identifies the bar size (number); while the last number represents the spliced length of the bar in inches. For example B-6-12 is a beam specimen reinforced with # 6 bars with 12 inches splice length. The average relative rib area for # 6 bars was found to be 0.1008 and for # 8 bars 0.90. To fully characterize the steel used in the tests, the mechanical and geometrical properties of the tested MMFX bars are reported in table (3.2). Yielding strength of the transverse reinforcement, based on an average value of three tests, is given in table (3.5).

3.3.3.2 Concrete:

Concrete was supplied by a local ready-mix company. The specimens in each group were cast at one time from the same batch of concrete to ensure consistency in the concrete properties. Except for the T-shaped beams which were cast one at a time due to the presence of one form work. The water-cement ratios (w/c) for the concrete used were in the range of 0.12 and 0.35. The fly ash content ranged from 5.19 lb/ft$^3$ to 6.94 lb/ft$^3$. Type I Portland cement, river sand and # 67 coarse aggregate stones were used in preparing the concrete mix. The sand content ranged from 41.11 lb/ft$^3$ to 49.10 lb/ft$^3$ while the coarse aggregate content was between 67.26 lb/ft$^3$ and 70.15 lb/ft$^3$. The slump ranged between 3 ½ in. and 5 ¾ in. Table (3.6) shows the mix proportions for each batch of concrete and the slump values. Compressive strength was determined based on an average of at least three 4 in. x 8 in. cylinders. The cylinders were cured side by side and in the same manner as the test.
specimens. The cylinders were tested at ages of 28 days and also on the day of testing. The concrete cylinders compressive strengths, as measured on the day of testing, ranged from 5710 psi to 6880 psi and are provided in details in table (3.7).

3.3.4 Test Setup

All the beams were tested as simply supported beams under two concentrated loads. The distance between the concentrated loads was 6 ft. for all 16 ft. long specimens. However for the 20 ft specimens the length of the constant moment zone was 8 ft. A closed loop MTS 450 kips capacity hydraulic actuator was used to apply the load. For the 16 ft long specimens, a stiff W12x65 HSS steel beam was used to equally distribute the actuator load to two concentrated loads to create a 6 ft constant moment region along the beam. However for the 20 ft long specimens, a (14 in. x 7 15/16 in. x 10 ft) W shaped built up steel beam was used to distribute the load in to two concentrated loads 8 ft apart. These two HSS steel distributing beams were hanged to the actuator plate with screws in way that their self weights will not be included in the load or bending moment calculations. All the specimens were supported on one roller and one pinned bearing plate. Each bearing plate was in turn supported on a stiff HSS steel beam that rests on the laboratory rigid concrete floor. A schematic for the test setup used is shown in figure (3.7).

3.3.5 Instrumentations

Electric resistance strain gages, LVDTs and Pi gages were all the instrumentations used in testing all the beams. All LVDTs and Pi gages were calibrated before starting the test.
The MTS actuator used to apply the load is calibrated for load and stroke values by the MTS Company. A typical configuration for the instrumentations that have been used for all the beams is defined in the following lines. Two, 6mm (0.24 in) gauge length, strain gages were installed on the external bottom surface of the MMFX rebar (facing the concrete cover). The location of the strain gages from the center line of the beam varied from one group to the other. In general the location of the strain gages was selected within the constant moment region and at least 6 inches away from the end of the splice zone. The distances between the strain gages and the center line of the beam for all the specimens are given in table (3.8). Four LVDTs were used to measure the vertical displacement of the beam. Two of them were placed at the mid span of the beam to monitor the mid span deflection. The other two were positioned at the support location to measure any deflection at the supporting system. This will give more accurate results for deflection measured at mid span. Two Pi gages, with 200 mm (8 in) gage length were mounted on the external surface of the concrete specimens to measure the concrete strain at the same locations. The two long Pi gages were clued to the top surface of the concrete specimen to measure the strain of the most compressed concrete fiber. A typical schematic showing the configuration of the different instrumentations used is given in figure (3.8).

3.3.6 Test Procedure

The load was continuously applied by the actuator through a stroke control option with a loading rate of 0.042 in/min. The cracks were mapped and the cracks widths were measured using crack comparator at concrete surface. The readings of the instrumentations
together with the actuator load and stroke were monitored and recorded by means of a MEGADAC data acquisition system at a rate of 1 scan/second. The test duration varied from one specimen to the other and ranged between 16 min. for beam B-6-12 and 48 min. for beam B-8-72.
Note: the values at the top are for the beams reinforced with # 4 MMFX bars and those at the bottom are for the one reinforced with # 8 bar

**Figure (3.1)** Typical Concrete Dimensions for the Beam End specimens
Figure (3.2) Details of Reinforcement of Beam End Specimens

**SECTION B-B**

- # 4 MMFX bar
- # 8 MMFX bar
- # 3 @ 9 in
- # 3 @ 3 in
- 9 in
- 2 # 6
- 4 in
- 2 1/2 in
- 80 in

**SECTION C-C**

- 1 1/2 in
- 2 # 6 Grade 60
- 20 in
- 1 1/2 in
- 4 # 8
- The MMFX bar

Typical Stirrup Dimensions

Note: Same note as in figure (3.1)
Elevation View of Bond Test Setup

**Figure (3.3a)** Elevation view showing the test setup used for all the cantilever beams
Plan View of Bond Test setup

**Figure (3.3b)** Plan view showing the Test Setup used for all the cantilever beams
Figure (3.4) Schematic showing the location of the different instrumentations used
Figure (3.5) Typical Concrete Dimensions for the Rectangular and
Figure (3.6) Typical Details of reinforcement for all the specimens

Note: the values at the top of the dimension line are for the 16 ft long beams and those at the bottom are for the 20 ft long beams
**Figure (3.7)** Elevation and Plan View for the Test setup used for all beams

<table>
<thead>
<tr>
<th>HSS Steel beam for applying load at 2 points</th>
<th>The actuator</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 in</td>
<td>6 in</td>
</tr>
<tr>
<td>4 ft – 6 in</td>
<td>6 ft</td>
</tr>
<tr>
<td>5 ft – 6 in</td>
<td>4 ft – 6 in</td>
</tr>
<tr>
<td>6 ft</td>
<td>5 ft – 6 in</td>
</tr>
</tbody>
</table>

**Roller Support**

**Pin Support**

**Lab. Concrete floor**

**Section A-A**

<table>
<thead>
<tr>
<th>HSS Steel beam to support the specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 ft</td>
</tr>
<tr>
<td>8 ft</td>
</tr>
<tr>
<td>10 ft</td>
</tr>
<tr>
<td>12 in</td>
</tr>
<tr>
<td>15 in</td>
</tr>
</tbody>
</table>

**Note:** Same note as in figure (3.6)
Figure (3.8) Schematic showing the location of the different instrumentations used
Table (3.1) the nominal dimensions of the specimens, the stirrup details and the target concrete compressive strength for the beam end specimens.

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>n</th>
<th>MMFX Bar Location during cast</th>
<th>Concrete Dimensions</th>
<th>Concrete Cover</th>
<th>Stirrups details</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>l_b (in)</td>
<td>L (in)</td>
<td>b (in)</td>
</tr>
<tr>
<td>B1</td>
<td>1</td>
<td>Bottom</td>
<td>14</td>
<td>80</td>
<td>14</td>
</tr>
<tr>
<td>B2</td>
<td>1</td>
<td>Top</td>
<td>14</td>
<td>80</td>
<td>14</td>
</tr>
<tr>
<td>B3</td>
<td>1</td>
<td>Bottom</td>
<td>14</td>
<td>80</td>
<td>14</td>
</tr>
<tr>
<td>B4</td>
<td>1</td>
<td>Bottom</td>
<td>34</td>
<td>80</td>
<td>14</td>
</tr>
</tbody>
</table>

where;
n: number of MMFX rebar
l_b: Bonded length
L: beam length
b: beam width
h: beam height
d: beam effective depth
d_b: MMFX bar diameter
C_so: the thickness of the clear side concrete cover (measured from the bar surface to the concrete surface)
C_b: the thickness of the clear bottom concrete cover
f_c': Concrete Compressive Strength
S: Average spacing of the stirrups
d_st: stirrups diameter
f_yt: minimum specified yield strength of steel stirrups
T: Tension force in the MMFX rebar at failure measured using load cell
Table (3.2) the mechanical and geometrical properties of MMFX rebars.

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Yield Strength (ksi)</th>
<th>Tensile Strength (ksi)</th>
<th>Elongation at failure (Strain)</th>
<th>Nominal Diameter (in)</th>
<th>Weight (lb/ft)</th>
<th>Rib Spacing (in.)</th>
<th>Rib Height Average (in.)</th>
<th>Relative Rib area</th>
</tr>
</thead>
<tbody>
<tr>
<td># 4</td>
<td>116.0</td>
<td>165.3</td>
<td>0.07656</td>
<td>0.50</td>
<td>0.65</td>
<td>0.3313</td>
<td>0.035</td>
<td>0.1119</td>
</tr>
<tr>
<td># 6</td>
<td>119.91</td>
<td>176.0</td>
<td>0.09218</td>
<td>0.75</td>
<td>1.73</td>
<td>0.5010</td>
<td>0.048</td>
<td>0.1008</td>
</tr>
<tr>
<td># 8</td>
<td>118.35</td>
<td>176.4</td>
<td>0.09689</td>
<td>1.00</td>
<td>2.64</td>
<td>0.6663</td>
<td>0.058</td>
<td>0.0913</td>
</tr>
</tbody>
</table>

**Elevation View**

**Plan View**
Table (3.3) concrete cylinders compressive strength measured on the day of testing for the beam end specimens.

<table>
<thead>
<tr>
<th>Specimens ID</th>
<th>Av. Density (lb/ft³)</th>
<th>cylinder # 1 (psi)</th>
<th>cylinder # 2 (psi)</th>
<th>cylinder # 3 (psi)</th>
<th>Average (psi)</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>145.3</td>
<td>5370</td>
<td>4840</td>
<td>5510</td>
<td>5240</td>
<td>353.4</td>
</tr>
<tr>
<td>B2</td>
<td>146.8</td>
<td>5410</td>
<td>4960</td>
<td>5290</td>
<td>5220</td>
<td>233.0</td>
</tr>
<tr>
<td>B3</td>
<td>143.5</td>
<td>5130</td>
<td>5660</td>
<td>4910</td>
<td>5230</td>
<td>385.6</td>
</tr>
<tr>
<td>B4</td>
<td>144.9</td>
<td>5080</td>
<td>5550</td>
<td>5320</td>
<td>5320</td>
<td>235.1</td>
</tr>
</tbody>
</table>
Table (3.4) the nominal dimensions of the specimens, the reinforcement details and the target concrete strength for the splice specimens

<table>
<thead>
<tr>
<th>Group #</th>
<th>Beam ID</th>
<th>Shape</th>
<th>n</th>
<th>$l_s$ (in)</th>
<th>L (ft)</th>
<th>b (in)</th>
<th>B (in)</th>
<th>h (in)</th>
<th>d (in)</th>
<th>$d_b$ (in)</th>
<th>$C_{so}$ (in)</th>
<th>$C_{si}$ (in)</th>
<th>$C_b$ (in)</th>
<th>$f_c'$ (psi)</th>
<th>N</th>
<th>S (in)</th>
<th>$d_{st}$ (in)</th>
<th>$f_{yt}$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>B-6-12</td>
<td>R</td>
<td>2</td>
<td>12</td>
<td>12</td>
<td>16</td>
<td>12</td>
<td>N/A</td>
<td>12</td>
<td>10.25</td>
<td>0.75</td>
<td>2.25</td>
<td>2.25</td>
<td>1.375</td>
<td>5000</td>
<td>0</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>II</td>
<td>B-6-24</td>
<td>R</td>
<td>2</td>
<td>24</td>
<td>12</td>
<td>16</td>
<td>12</td>
<td>N/A</td>
<td>14</td>
<td>12.25</td>
<td>0.75</td>
<td>2.25</td>
<td>2.25</td>
<td>1.375</td>
<td>5000</td>
<td>0</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>III</td>
<td>B-6-36</td>
<td>T</td>
<td>2</td>
<td>36</td>
<td>16</td>
<td>12</td>
<td>24</td>
<td>18</td>
<td>16.25</td>
<td>0.75</td>
<td>2.25</td>
<td>2.25</td>
<td>1.375</td>
<td>5000</td>
<td>0</td>
<td>N/A</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>IV</td>
<td>B-6-60</td>
<td>T</td>
<td>2</td>
<td>60</td>
<td>20</td>
<td>12</td>
<td>48</td>
<td>18</td>
<td>16.25</td>
<td>0.75</td>
<td>2.25</td>
<td>2.25</td>
<td>1.375</td>
<td>5000</td>
<td>0</td>
<td>N/A</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>B-8-12</td>
<td>R</td>
<td>1</td>
<td>12</td>
<td>16</td>
<td>12</td>
<td>N/A</td>
<td>12</td>
<td>10.13</td>
<td>1.00</td>
<td>5.00</td>
<td>N/A</td>
<td>1.375</td>
<td>5000</td>
<td>1</td>
<td>12</td>
<td>0.375</td>
<td>60</td>
</tr>
<tr>
<td>II</td>
<td>B-8-24</td>
<td>R</td>
<td>1</td>
<td>24</td>
<td>16</td>
<td>12</td>
<td>N/A</td>
<td>14</td>
<td>12.13</td>
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<td>N/A</td>
<td>1.375</td>
<td>5000</td>
<td>2</td>
<td>12</td>
<td>0.375</td>
<td>60</td>
</tr>
<tr>
<td>III</td>
<td>B-8-48</td>
<td>T</td>
<td>1</td>
<td>48</td>
<td>16</td>
<td>12</td>
<td>24</td>
<td>18</td>
<td>16.13</td>
<td>1.00</td>
<td>5.00</td>
<td>N/A</td>
<td>1.375</td>
<td>5000</td>
<td>5</td>
<td>9.6</td>
<td>0.375</td>
<td>60</td>
</tr>
<tr>
<td>IV</td>
<td>B-8-72</td>
<td>T</td>
<td>1</td>
<td>72</td>
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<td>48</td>
<td>18</td>
<td>16.13</td>
<td>1.00</td>
<td>5.00</td>
<td>N/A</td>
<td>1.375</td>
<td>5000</td>
<td>7</td>
<td>10.3</td>
<td>0.375</td>
<td>60</td>
</tr>
</tbody>
</table>

where;
- n: number of Spliced bars
- $l_s$: splice length
- L: beam length
- b: beam web width
- B: beam flange width
- h: beam height
- d: beam effective depth
- $d_b$: Spliced bar diameter
- $C_{so}$: the thickness of the clear side concrete cover (measured from the bar surface to the concrete surface)
- $C_{si}$: half the clear spacing between the spliced bars
- $C_b$: the thickness of the clear bottom concrete cover
- $f_c'$: Concrete Compressive Strength
- N: Total number of stirrups along the splice length
- S: Average spacing of the stirrups
Table (3.5) the measured dimensions of the specimens, the stirrups details and the cylinders concrete strength as measured on the day of testing for the splice specimens.

<table>
<thead>
<tr>
<th>Group #</th>
<th>Specimen ID</th>
<th>l_s (in)</th>
<th>L (ft)</th>
<th>b (in)</th>
<th>B (in)</th>
<th>h (in)</th>
<th>d (in)</th>
<th>C_so (in)</th>
<th>C_si (in)</th>
<th>C_b (in)</th>
<th>f_y (psi)</th>
<th>N</th>
<th>S (in)</th>
<th>d_st (in)</th>
<th>f_yt (ksi)</th>
<th>P (kips)</th>
<th>M (kips-in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>B-6-12</td>
<td>12</td>
<td>16.00</td>
<td>12.00</td>
<td>N/A</td>
<td>12.25</td>
<td>10.38</td>
<td>2.375</td>
<td>2.125</td>
<td>1.500</td>
<td>5800</td>
<td>0</td>
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<td>N/A</td>
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<td>517</td>
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<td>24</td>
<td>16.00</td>
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<td>2.750</td>
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<td>2.125</td>
<td>2.375</td>
<td>1.375</td>
<td>6370</td>
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<td>B-6-60</td>
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<td>20.00</td>
<td>12.00</td>
<td>48.00</td>
<td>18.00</td>
<td>16.25</td>
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<td>2.250</td>
<td>1.375</td>
<td>5710</td>
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<td>16.00</td>
<td>12.25</td>
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<td>18.5</td>
<td>16.63</td>
<td>4.500</td>
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<td>6880</td>
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<td>12.00</td>
<td>48.00</td>
<td>18.00</td>
<td>16.13</td>
<td>4.750</td>
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<td>1.375</td>
<td>5940</td>
<td>7</td>
<td>10.25</td>
<td>0.375</td>
<td>63</td>
<td>46.5</td>
<td>1716</td>
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</tbody>
</table>

* The dimensions of the specimens were measured after casting

** The concrete covers, bar spacing, and stirrups spacing were measured before casting and after testing and the most reliable values are reported

P: Applied load by the actuator at the time of bond failure

M: Calculated bending moment at the time of bond failure at the splice location including the effect of the specimen self weight

d_s: stirrups diameter

f_y: minimum specified yield strength of steel stirrups

N/A : Not applicable

R: Rectangular cross section

T: T-shaped cross section
Table (3.6) concrete mix proportions and slump values for splice specimens.

<table>
<thead>
<tr>
<th>Group #</th>
<th>Specimen ID</th>
<th>w/cm Ratio</th>
<th>Cement (lb/ft³)</th>
<th>Water (lb/ft³)</th>
<th>Fine Agg. (lb/ft³)</th>
<th>Coarse Agg. (lb/ft³)</th>
<th>Fly Ash (lb/ft³)</th>
<th>Air entertaining agent (oz/yd³)</th>
<th>Water retarder (oz/yd³)</th>
<th>Slump (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>B-6-12</td>
<td>0.12</td>
<td>25.71</td>
<td>2.97</td>
<td>45.32</td>
<td>70.15</td>
<td>N/A</td>
<td>2.8</td>
<td>21</td>
<td>5.75</td>
</tr>
<tr>
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<td>B-8-12</td>
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<td></td>
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<td></td>
</tr>
<tr>
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<td>B-6-24</td>
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<td>23.92</td>
<td>7.00</td>
<td>49.10</td>
<td>68.57</td>
<td>5.29</td>
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<td>4.5</td>
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<tr>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>III</td>
<td>B-6-36</td>
<td>0.35</td>
<td>23.52</td>
<td>4.88</td>
<td>41.11</td>
<td>67.77</td>
<td>6.94</td>
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<tr>
<td></td>
<td>B-8-72</td>
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<td>23.26</td>
<td>5.27</td>
<td>42.67</td>
<td>67.26</td>
<td>6.14</td>
<td>N/A</td>
<td>25</td>
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Table (3.7) concrete cylinders compressive strength as measured on the day of testing for splice specimens.

<table>
<thead>
<tr>
<th>Group #</th>
<th>Specimen ID</th>
<th>Av. Density (lb/ft³)</th>
<th>cylinder # 1 (psi)</th>
<th>cylinder # 2 (psi)</th>
<th>cylinder # 3 (psi)</th>
<th>Average (psi)</th>
<th>Standard Deviation</th>
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<tr>
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<td>142</td>
<td>5810</td>
<td>5490</td>
<td>6110</td>
<td>5800</td>
<td>310.0</td>
</tr>
<tr>
<td></td>
<td>B-8-12</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
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<td>B-6-24</td>
<td>142.4</td>
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<td>5830</td>
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<td>5720</td>
<td>359.0</td>
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<td></td>
</tr>
<tr>
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<td>6480</td>
<td>5870</td>
<td>6370</td>
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<td>6900</td>
<td>6710</td>
<td>6880</td>
<td>166</td>
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<td>5710</td>
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<td>5780</td>
<td>6090</td>
<td>-</td>
<td>5940</td>
<td>219.3</td>
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Table (3.8) distances between strain gages and center line of the specimen.

<table>
<thead>
<tr>
<th>Group #</th>
<th>Specimen ID</th>
<th>X (in.)</th>
</tr>
</thead>
<tbody>
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<tr>
<td></td>
<td>B-8-12</td>
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<td>B-6-24</td>
<td>24</td>
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<tr>
<td></td>
<td>B-8-24</td>
<td></td>
</tr>
<tr>
<td>III</td>
<td>B-6-36</td>
<td>30</td>
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<td></td>
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<tr>
<td>IV</td>
<td>B-6-60</td>
<td>42</td>
</tr>
<tr>
<td></td>
<td>B-8-72</td>
<td></td>
</tr>
</tbody>
</table>

X : distance between strain gages and center line of the beam (refer to figure 3.4)
4.1 Introduction

In this chapter the experimental results of the two phases of investigation are presented. The test results for each program are presented separately in both a tabular and graphical format. The results include behavior of the tested specimens at each limit state including the observations and the readings of the different instrumentations.

4.2 Test program I: Beam End Specimens

4.2.1 General

This section presents the measured test results for the four beams tested in program I. The results of each beam will be presented separately. The presentation will include the observation and the remarks that were recorded during the test including the cracks properties and the mode of failure. Experimental graphs showing the response of the beams during loading will be provided. Actual photos for the tested beams, taken during and after testing, are provided to help for a better understanding for the beams performance.

4.2.2 Beam B1

This beam was reinforced with one # 4 MMFX rebar, bottom cast, with a 14 inch bonded length and # 3 stirrups every 9 in. As the beam was loaded, flexural cracks developed along the bonded length of the MMFX bar. As the applied load increased, the number and width of these cracks increased. The average spacing between the flexural cracks was
approximately equal to the spacing between stirrups. Failure occurred at a load level of 33.8 kips by the rupture of the MMFX rebar after reaching its tensile strength. No signs of bond failure were observed. The curve showing the tensile force in the MMFX bar versus the measured slip at the loaded and unloaded ends of the rebar is given in figure (4.1). Figure (4.2) shows the relationship between the tensile stresses developed in the MMFX steel rebars, as measured by the load cell, and the strains, as measured by the strain gages and the extensometer.

4.2.3 Beam B2

This beam was reinforced with one # 4 MMFX rebar, top cast, with a 14 inch bonded length and # 3 stirrups every 9 in. As the beam was loaded, flexural cracks developed along the bonded length of the MMFX bar. As the applied load increased, the number and width of these cracks increased. The flexural cracks pattern as observed during the test is shown in figure (4.3). Failure occurred at a load level of 32.3 kips by the rupture of the MMFX rebar after reaching its tensile strength. No signs of bond failure were observed. The curve showing the tensile force developed in the MMFX bar versus the measured slip at the loaded and unloaded ends of the rebar is given in figure (4.4). Figure (4.5) shows the relationship between the stresses that developed in the MMFX steel rebars, as measured by the load cell, and the strains as measured by the strain gages and the extensometer.

4.2.4 Beam B3

This beam was reinforced with one # 4 MMFX rebar, bottom cast, with a 14 inch
bonded length and # 3 stirrups every 3 in. As the beam was loaded, flexural cracks developed along the bonded length of the MMFX bar. As the applied load increased, the number and width of these cracks increased. The average spacing between the flexural cracks was approximately equal to the spacing between stirrups. Failure occurred at a load level of 32.4 kips by the rupture of the MMFX rebar after reaching its tensile strength. No signs of bond failure were observed. The curve showing the axial tensile force induced in the MMFX bar, as measured by the load cell, versus the measured slip at the loaded and unloaded ends of the rebar is given in figure (4.6). Figure (4.7) shows the relationship between the stresses that developed in the MMFX steel rebars and the strains as measured by the strain gages and the extensometer.

4.2.5 Beam B4

This beam was reinforced with one # 8 MMFX rebar, bottom cast, with a 34 inch bonded length and # 3 stirrups every 9 in. As the beam was loaded, flexural cracks developed along the bonded length of the MMFX bar. As the applied load increased, the number and width of these cracks increased. The average spacing between the flexural cracks was equal to the spacing between stirrups. A longitudinal crack developed at the loaded end and started to propagate towards the unloaded end running through the flexural cracks and along the bottom cover as shown in figure (4.8). Failure occurred at a load level of 90.8 kips by the splitting of the bottom concrete cover. A splitting mode of bond failure was observed. There was also an excessive increase in the width of the first flexural crack near the loaded end as shown in figure (4.8). The curve showing the tensile force developed in the MMFX bar
versus the measured slip at the loaded and unloaded ends of the rebar is given in figure (4.9).

Figure (4.10) shows the relation between the stresses that developed in the MMFX steel rebars as measured by the load cell and the strains as measured by the strain gages and the extensometer.

4.3 Test program II: Splice Specimens

4.3.1 General

This section sums the measured test results for the eight beams tested in the second program. The outcomes of each beam will be presented separately. The presentation will include the observation and the remarks that were recorded during the test including the cracks properties and the mode of failure. Experimental graphs showing the performance of the tested beams under loading will be shown. Photos of the tested beams, taken during and after testing, are provided to help for a better understanding for the beams structural response.

4.3.2 Beam B-8-12

This beam was reinforced by one lapped spliced # 8 MMFX rebar with a 12 in. splice length. The first two flexural cracks were observed within the constant moment zone but away from the splice length zone at load level of 3.6 kips. As the load continued to increase more flexural cracks started to develop. Two main flexural cracks were expected to develop at the ends of the spliced bars. At a load level of 5.3 kips, these two main flexural cracks formed very close to the ends of the spliced length. The rate of increase in the width of these
two main cracks with load was recorded at different load levels and presented in table (4.1) and figure (4.11). Two longitudinal splitting cracks appeared on the bottom concrete cover starting from the two main flexural cracks and running towards the mid-span. These two longitudinal cracks propagated towards each other as the load increased. At a load level of 11.2 kips, a sudden and excessive increase in the width of one of the two main cracks was observed indicating slippage of MMFX rebar, see figure (4.12). Along the bottom surface of the beam, a typical splitting bond failure crack pattern was seen at the splice location, as shown in figure (4.13). These observations were accompanied by a significant drop in the applied load indicating a typical splitting bond failure mechanism in the splice. The failure was, also, characterized by being contained without any spalling of concrete covers. Figure (4.14) shows the applied load versus the measured mid-span deflection. Figure (4.15) presents the relationship between the applied load and the strain that developed in the MMFX rebar, as measured by the strain gages. The relationship between the applied load and the concrete strain at the extremely compressed fiber, as measured by the Pi gages, is given in figure (4.16).

4.3.3 Beam B-6-12

This beam was reinforced by two lapped spliced # 6 MMFX rebars with a 12 in. splice length. The first two flexural cracks were observed within the constant moment zone but away from the splice zone at load level of 4.0 kips. As the load continued to increase more flexural cracks started to develop. The cracks were marked and the crack pattern developed as the load increased. The photo, shown in figure (4.17), shows part of the crack
pattern as mapped on the beam side. Two main flexural cracks were expected to develop at the ends of the spliced bars. At a load level of 6.0 kips, the two main cracks were formed very close to the ends of the spliced length. The two cracks continued to widen as the load increased. The increase in the width of these two cracks with load was measured using crack comparator and presented in table (4.2) and figure (4.18). Two longitudinal splitting cracks developed on the bottom concrete cover starting from the two main flexural cracks and running towards the mid-span. These two longitudinal cracks propagated towards each other as the load increased. At a load level of 30.4 kips, a sudden and excessive increase in the width of one of the two main flexural cracks was observed causing a deep propagation of these cracks towards the top surface of the beam. Also, two diagonal cracks running across the two primary flexural cracks developed on the beam side as seen in figure (4.19). The slippage of the spliced MMFX rebar was accompanied by a significant drop in the applied load indicating a typical splitting bond failure mechanism in the splice. Along the bottom surface of the beam, a typical splitting bond failure crack pattern was seen at the splice location, as shown in figure (4.20). The bond failure, also, experienced spalling and scattering of the bottom cover concrete due to the absence of stirrups along the splice length. After removing part of the concrete cover, the concrete surface in contact with the MMFX rebar was examined and no signs of concrete crushing were observed as seen in figures (4.21a) and (4.21b). Figure (4.22) shows the applied load versus the measured mid-span deflection. Figure (4.23) shows the relationship between the applied load and the strains that developed in the MMFX steel rebars as measured by the strain gages. The relationship between the load and the concrete strain at the extremely compressed fiber, as given by the Pi
gages, is presented in figure (4.24).

4.3.4 Beam B-8-24

This beam was reinforced by one lapped spliced # 8 MMFX rebar with a 24 in. splice length. The first two flexural cracks were observed within the constant moment zone but away from the splice length zone at load level of 3.6 kips. As the load continued to increase more flexural cracks started to develop. Two main flexural cracks were expected to develop at the ends of the spliced bars. At a load level of 5.3 kips, these two flexural cracks formed close to the two ends of the splice length. The rate of increase in the width of these two cracks with load was recorded at different load levels and presented in table (4.3) and figure (4.25). Two longitudinal splitting cracks appeared on the bottom concrete cover starting from the two main flexural cracks and running towards the mid-span. These two longitudinal cracks propagated towards each other as the load increased. At a load level of 19.8 kips, a sudden and excessive increase in the width of one of the two main cracks was observed indicating slippage of MMFX rebar, see figure (4.26). Along the bottom surface of the beam, a typical splitting bond failure crack pattern was seen at the splice location, as shown in figure (4.27). These observations were accompanied by a significant drop in the applied load indicating a typical splitting bond failure mechanism in the splice. The failure was, also, characterized by being contained without any spalling of concrete covers. Figure (4.28) shows the applied load versus the measured mid-span deflection. Figure (4.29) presents the relationship between the applied load and the strain that developed in the MMFX rebar, as measured by the strain gages. The relationship between the applied load and the concrete
strain at the extremely compressed fiber, as measured by the Pi gages, is given in figure (4.30).

4.3.5 Beam B-6-24

This beam was reinforced by two lapped spliced # 6 MMFX rebars with a 24 in. splice length. The first two flexural cracks were observed within the constant moment zone but away from the splice length zone at load level of 6.5 kips. As the load continued to increase more flexural cracks started to develop. The cracks were marked and the crack pattern progressively developed. The photo shown in figure (4.31) shows the mapped crack pattern on the beam side during loading. Two main flexural cracks were expected to develop at the ends of the spliced bars. At a load level of 10.8 kips, one of them was formed close to the right end of the splice length, while the other crack formed at a load level of 17 kips. The two cracks continued to widen as the load increased. The increase in the width of these two cracks with load was measured very load increment using a crack comparator and is presented in table (4.4) and figure (4.32). Two longitudinal splitting cracks developed on the bottom concrete cover starting from the two main flexural cracks and running towards the mid-span. These two longitudinal cracks propagated towards each other as the load increased. At a load level of 30.4 kips, a sudden and excessive increase in the width of the two main flexural cracks was observed causing a deep propagation of these cracks towards the top surface of the beam, as seen in figure (4.33). These observations were accompanied by a significant drop in the applied load indicating a typical splitting bond failure mechanism in the splice. Along the bottom surface of the beam, a typical splitting bond failure crack
pattern was noticed at the splice location. The bond failure, also, experienced spalling and scattering of the concrete covers due to the absence of stirrups along the splice length. Due to this excessive spalling and scattering of the side and bottom concrete covers at failure, the splice lost all its strength, leading to concrete crushing under the beam self weight as shown in figure (4.34). After removing part of the concrete cover, the concrete surface in contact with the MMFX rebar was examined and no signs of concrete crushing were observed. Figure (4.35) shows the applied load versus the measured mid-span deflection. Figure (4.36) shows the relationship between the applied load and the strains that developed in the MMFX steel rebars as measured by the strain gages. The relationship between the load and the concrete strain at the extremely compressed fiber, as given by the Pi gages, is presented in figure (4.37).

4.3.6 **Beam B-8-48**

This beam had a T-shaped cross section and was reinforced by one lapped spliced # 8 MMFX rebar with a 48 in. splice length. The first flexural crack was observed within the splice length zone at load level of 6.0 kips. As the load continued to increase more flexural cracks started to develop. Two main flexural cracks were expected to develop at the ends of the spliced bars. The first crack was one of them. The other crack formed at a load level of 12 kips. The rate of increase in the width of these two cracks with load was recorded at different load levels and presented in table (4.5) and figure (4.38). Two longitudinal splitting cracks appeared on the bottom concrete cover starting from the two main flexural cracks and propagating towards the mid-span. These two longitudinal cracks spread towards each other.
as the load increased forming a crack mesh. At a load level of 51.5 kips, a sudden and significant drop in the applied load occurred without experiencing excessive increase in the width of any of the two main cracks, see figure (4.39). Along the bottom surface of the beam, a typical splitting bond failure crack pattern was observed at the splice location, as shown in figure (4.40). These observations were accompanied by a drop in the applied load indicating a typical splitting bond failure mechanism in the splice. The failure was, also, characterized by being contained without any spalling of concrete covers. Figure (4.41) shows the applied load versus the measured mid-span deflection. Figure (4.42) presents the relationship between the applied load and the strain that developed in the MMFX rebar, as measured by the strain gages. The relationship between the applied load and the concrete strain at the extremely compressed fiber, as measured by the Pi gages, is given in figure (4.43).

4.3.7 Beam B-6-36

This beam was reinforced by two lapped spliced # 6 MMFX rebars with a 36 in. splice length. The first two flexural cracks were observed within the splice length zone at load level of 13.5 kips. As the load continued to increase more flexural cracks started to develop. The cracks were marked and the crack pattern progressively developed. The photo shown in figure (4.44) shows the mapped crack pattern on the beam side during loading. Two main flexural cracks were expected to develop at the ends of the spliced bars. At a load level of 17 kips, these two cracks developed. The two cracks continued to widen as the load increased. The increase in the width of these two cracks with load was measured at different load levels using a crack comparator and is presented in table (4.6) and figure (4.45). Two
longitudinal splitting cracks developed on the bottom concrete cover starting from the two main flexural cracks and running towards the mid-span. These two longitudinal cracks propagated towards each other as the load increased. At a load level of 55.5 kips, a sudden and excessive increase in the width of one of the two main flexural cracks was observed causing a deep propagation of these cracks towards the flange of the beam, as seen in figure (4.46). These observations were accompanied by a significant drop in the applied load indicating a typical splitting bond failure mechanism in the splice. Along the bottom surface of the beam, a typical splitting bond failure crack pattern was noticed at the splice location; refer to figure (4.47). The bond failure, also, experienced spalling and scattering of the concrete covers due to the absence of stirrups along the splice length. After removing part of the concrete cover, the concrete surface in contact with the MMFX rebar was examined and no signs of concrete crushing were observed. Figure (4.48) shows the applied load versus the measured mid-span deflection. Figure (4.49) shows the relationship between the applied load and the strains that developed in the MMFX steel rebars as measured by the strain gages. The relationship between the load and the concrete strain at the extremely compressed fiber, as given by the Pi gages, is presented in figure (4.50).

4.3.8 Beam B-8-72

This beam had a T-shaped cross section and was reinforced by one lapped spliced # 8 MMFX rebar with a 72 in. splice length. The first flexural crack was observed within the splice length zone at load level of 6.0 kips. As the load continued to increase more flexural cracks started to develop. Two main flexural cracks were expected to develop at the ends of
the spliced bars. The first crack was one of them. The other crack formed at a load level of 13 kips. The rate of increase in the width of these two cracks with load was recorded at different load levels and presented in table (4.7) and figure (4.51). Two longitudinal splitting cracks appeared on the bottom concrete cover starting from the two main flexural cracks and propagating towards the mid-span. These two longitudinal cracks spread towards each other as the load increased forming a crack mesh. As we get close to the failure load, diagonal cracks running on the side of the beam and meeting with the pre-developed flexural cracks were noticed, refer to figure (4.52). At a load level of 46.3 kips, a sudden and significant drop in the applied load occurred but without showing excessive increase in the width of any of the two main cracks, see figure (4.53). Along the bottom surface of the beam, a typical splitting bond failure crack pattern was seen at the splice location, as shown in figure (4.54). These observations were accompanied by a drop in the applied load indicating a typical splitting bond failure mechanism in the splice. The failure was, also, characterized by being contained without any spalling of concrete covers. Figure (4.55) shows the applied load versus the measured mid-span deflection. Figure (4.56) presents the relationship between the applied load and the strain that developed in the MMFX rebar, as measured by the strain gages. The relationship between the applied load and the concrete strain at the extremely compressed fiber, as measured by the Pi gages, is given in figure (4.57).

4.3.9 Beam B-6-60

This beam was reinforced by two lapped spliced # 6 MMFX rebars with a 60 in. splice length. The first two flexural cracks were observed within the splice length zone at
load level of 7.0 kips. As the load continued to increase more flexural cracks started to develop. The cracks were marked and the crack pattern progressively developed. The photo shown in figure (4.58) shows the mapped crack pattern on the beam side during loading. Two main flexural cracks were expected to develop at the ends of the spliced bars. At a load level of 15 kips, these two cracks developed. The two cracks continued to widen as the load increased. The increase in the width of these two cracks with load was measured at different load levels using a crack comparator and is presented in table (4.8) and figure (4.59). Two longitudinal splitting cracks developed on the bottom concrete cover starting from the two main flexural cracks and running towards the mid-span. These two longitudinal cracks propagated towards each other as the load increased. As we get close to the failure load, diagonal cracks running on the side of the beam and meeting with the pre-developed flexural cracks were noticed, refer to figure (4.58). At a load level of 45.2 kips, a sudden and excessive increase in the width of one of the two main flexural cracks was observed causing a deep propagation of these cracks towards the flange of the beam, as seen in figure (4.60). These observations were accompanied by a significant drop in the applied load indicating a typical splitting bond failure mechanism in the splice. Along the bottom surface of the beam, a typical splitting bond failure crack pattern was noticed at the splice location; refer to figure (4.60). The bond failure, also, experienced spalling and scattering of the concrete covers due to the absence of stirrups along the splice length. After removing part of the concrete cover, the concrete surface in contact with the MMFX rebar was examined and no signs of concrete crushing were observed. Figure (4.61) shows the applied load versus the measured mid-span deflection. Figure (4.62) shows the relationship between the applied load and the strains that
developed in the MMFX steel rebars as measured by the strain gages. The relationship between the load and the concrete strain at the extremely compressed fiber, as given by the Pi gages, is presented in figure (4.63).
Figure (4.1) Applied Load vs. Loaded and Unloaded End Slip for B1
Figure (4.2) Stress vs. Strain for B1
Figure (4.3) Flexural cracks distribution along the bottom cover with no signs of splitting longitudinal cracks at time of bond failure for B1
Figure (4.4) Applied Load vs. Loaded and Unloaded End Slip for B2
Figure (4.5) Stress vs. Strain for B2

- Strain gage D1
- Strain gage D2
- Strain gage D3
- Extensometer
Figure (4.6) Applied Load vs. Loaded and Unloaded End Slip for B3
Figure (4.7) Stress vs. Strain for B3
Figure (4.8) Splitting cracks propagation through the bottom cover at time of bond failure for B4
Figure (4.9) Applied Load vs. Loaded and Unloaded End Slip for B4
Figure (4.10) Stress vs. Strain for B4
Figure (4.11) Applied Load vs. Crack width for B-8-12
Figure (4.12) Bar slippage at the time of bond failure for B-8-12
Figure (4.13) Longitudinal splitting crack running between the two flexural cracks for B-8-12
Figure (4.14) Applied Load vs. Mid-span Deflection for B-8-12
Figure (4.15) Applied Load vs. MMFX Rebar Strain for B-8-12
Figure (4.16) Applied Load vs. Concrete Compression Strain for B-8-12
Figure (4.17) Mapped crack pattern on the side of beam B6-12 during loading
Figure (4.18) Applied Load vs. Crack width for B-6-12
Figure (4.19) Two diagonal cracks running between the two main flexural cracks Beam B-8-12

Splice length = 12 in.
Splitting cracks propagation along the bottom surface of the beam

**Figure (4.20)** Typical splitting bond failure crack pattern on the bottom surface of Beam B-6-12
Figure (4.21) No signs of concrete crushing were observed on the concrete keys surrounding the MMFX rebar in Beam B-6-12
Figure (4.22) Applied Load vs. Mid-span Deflection for B-6-12
Figure (4.23) Applied Load vs. MMFX Rebar Strain for B-6-12

X: malfunctioning after this point
Figure (4.24) Applied Load vs. Concrete Compression Strain for B-6-12
Figure (4.25) Applied Load vs. Crack width for B-8-24
Figure (4.26) A sudden and excessive increase in the width of one of the two main cracks just after failure in Beam B-8-24
Figure (4.27) A typical splitting bond failure crack pattern on the bottom surface of the specimen in Beam B-8-24
Figure (4.28) Applied Load vs. Mid-span Deflection for B-8-24
Figure (4.29) Applied Load vs. MMFX Rebar Strain for B-8-24
Figure (4.30) Applied Load vs. Concrete Compression Strain for B-8-24
Figure (4.31) mapped crack pattern on the side of beam B-6-24 during loading
Figure (4.32) Applied Load vs. Crack width for B-6-24
Figure (4.33) excessive and sudden increases in crack width at failure and concrete spalling at failure in beam B-6-24
Figure (4.34) concrete crushing and complete failure of the beam B-6-24 under self weight after splice failure
**Figure (4.35)** Applied Load vs. Mid-span Deflection for B-6-24
**Figure (4.36)** Applied Load vs. MMFX Rebar Strain for B-6-24
Figure (4.37) Applied Load vs. Concrete Compression Strain for B-6-24
Figure (4.38) Applied Load vs. Crack width for B-8-48
Figure (4.39) no excessive increase in the width of the flexural crack at the end of the splice length was seen at time of failure in Beam B-8-48
Figure (4.40) a typical splitting bond failure crack pattern was observed at the splice location at time of bond failure in Beam B-8-48
Figure (4.41) Applied Load vs. Mid-span Deflection for B-8-48
Figure (4.42) Applied Load vs. MMFX Rebar Strain for B-8-48
Figure (4.43) Applied Load vs. Concrete Compression Strain for B-8-48
Figure (4.44) mapping the crack pattern on the side of beam B-6-36 during loading
Figure (4.45) Applied Load vs. Crack width for B-6-36
Figure (4.46) sudden and excessive increases in the width of one of the cracks at the time of bond failure in beam B-6-36
Figure (4.47) typical splitting bond failure mechanism on the bottom surface of beam B-6-36 observed just at failure
Figure (4.48) Applied Load vs. Mid span Deflection for B-6-36
Figure (4.49) Applied Load vs. MMFX Rebar Strain for B-6-36
Figure (4.50) Applied Load vs. Concrete Compression Strain for B-6-36
Figure (4.51) Applied Load vs. Crack width for B-8-72
Figure (4.52) Mapped crack pattern on the side of beam B-8-72 during loading
Figure (4.53) No excessive increase in the width of the flexural crack at the end of the splice length was seen at time of failure in beam B-8-72
Figure (4.54) A typical splitting bond failure crack pattern was observed at the splice location at time of bond failure in beam B-8-72
Figure (4.55) Applied Load vs. Mid-span Deflection for B-8-72
Figure (4.56) Applied Load vs. MMFX Rebar strain for B-8-72
Figure (4.57) Applied Load vs. Concrete Compression Strain for B-8-72
Figure (4.58) Crack pattern with diagonal cracks running on the side of beam B-6-60 and meeting with the flexural cracks during loading
Figure (4.59) Applied Load vs. Crack Width for B-6-60
Figure (4.60) excessive increases in the width of one of the two main flexural cracks at bond failure with spalling of bottom and side covers in beam B-6-60
Figure (4.61) Applied Load vs. Mid-span Deflection for B-6-60
Figure (4.62) Applied Load vs. MMFX Rebar Strain for B-6-60
Figure (4.63) Applied Load vs. Concrete Compression Strain for B-6-60
Table (4.1) the 2 primary cracks widths as measured during loading for B-8-12

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Table (4.2) the 2 primary cracks widths as measured during loading for B-6-12

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Table (4.3) the 2 primary cracks widths as measured during loading for B-8-24

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Table (4.4) the 2 primary cracks widths as measured during loading for B-6-24

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Table (4.5) the 2 primary cracks widths as measured during loading for B-8-48

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### Table (4.8) the 2 primary cracks widths as measured during loading for B-6-60

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5.1 Introduction

This chapter presents analysis of the experimental results obtained from the two phases of the experimental program; point out their significance and to draw out some observations. The relationship between the different measurements, presented in chapter 4, will be discussed and closely examined. Test results will be compared to predicted values using the descriptive and the design expressions presented in chapter 2.

5.2 Test Program I: Beam End Specimens

5.2.1 General

The experimental outcomes of the first test program that included testing four beam end specimens will be examined. Some observations will be pointed out from the results. The different mechanisms based on these observations will be provided.

5.2.2 Discussion and Comparison of the Bond Behavior

By examining the four stress strain relationships, shown in chapter 4, figures (4.2), (4.5), (4.7), and (4.10) for the four beam end specimens, where the bar stresses are determined using the load cell readings and the nominal bar diameter; and the strains are measured using the extensometer mounted on the bar free end, the following observations are made. The proportional limit strength, as determined from the stress strain curves, differed from one beam to the other. The values of the proportional limit strength for B1, B2, B3, and B4, measured from the beam response were 100 ksi, 70 ksi, 60 ksi, and 50 ksi respectively.
Most of these proportional limit stress values are below the average value obtained from the mechanical properties of the bar which is around 90 ksi as mentioned in chapter 2, except for B1 which gave a quite close value using the mechanical properties of the bar. In general this dissimilarity could be due to misalignment or imperfection in the installation of the extensometer, the influence of the change in the loading rate from that used to determine the tensile strength characteristics on the stress-strain response. Also, the elastic moduli, \( E_s \), for MMFX rebar as measured from these curves were about 29,000 ksi for both B1 and B3, however, for B2 and B4 the elastic moduli were 24000 ksi and 35000 ksi respectively. Again such deviation of the elastic moduli values from the 29000 ksi values obtained from the mechanical investigation could be explained on the same basis of misalignment and difference between the nominal and actual bar diameter mentioned above.

Having the test conducted under a load control option, the full stress-strain curve of the MMFX rebar, which includes the softening portion, was not observed. In addition, the extensometer was removed, in all the beams except for B4, before reaching the ultimate tensile strength of the MMFX to protect the extensometer from damage. The strains measured by the three strain gages installed on the rebar surface along the bonded length seem reasonable in their response. This is evident because the values of the measured strain by the three strain gages were influenced by the location of the strain gage from the loading end.

Figures (5.1), (5.2), and (5.3) show the bond stress distribution along the bonded length for beams B1, B2, and B3 at different stress levels, namely 80 ksi, 100 ksi, and 120 ksi respectively. The bond stress distribution was determined using the following procedure.
At a given bar stress level, the corresponding strain readings along the bonded length as measured by the three strain gages are determined. Using the stress-strain curve obtained from the mechanical properties of the bar, the corresponding stress levels were obtained for each strain gage reading along the bar. These stress values were linearly connected assuming a linear stress distribution along the bonded length. Based on the linear assumption, the bond stress between any two successive strains is assumed to be a constant. Assuming zero values for bond stress at the beginning and at the end of the bonded length of the bar, the average bond stress between every two successive strain gages were computed using the following equation:

$$u_{bi} = \frac{(f_i - f_{i-1})d_b}{4(x_{i-1} - x_i)}$$

Equation (5.1)

Where:

- $u_{bi}$: is the average bond stress between strain $(i)$ and $(i-1)$.
- $f_i$: is the bar stress at the location of the strain gage $(i)$.
- $f_{i-1}$: is the bar stress at the location of the next strain gage $(i-1)$.
- $d_b$: is the bar nominal diameter.
- $x_i$: is the distance between the loaded end and the strain gage $(i)$.
- $x_{i-1}$: is the distance between the loaded end and the strain gage $(i-1)$.

By examining all the bond stress distribution curves, it is clear that the bond stress distribution is essentially non-uniform and the peak value for bond stress is located near the loaded end. The pattern obtained is typical to the behavior of conventional steel rebars, Lutz.
and Gergely [1967] and Azizinamini et al. [1994, 1999], however the bond stress values are relatively lower than the ultimate bond strength.

At stress level of 80 ksi, shown in figure (5.1), by comparing the average bond stress distribution of the MMFX bars used as top reinforcement for beam B2, and the bars used as bottom reinforcement for beam B1, it is obvious that the bond stress distribution in B2 is more uniformly distributed than that in B1. This is evident from the smaller value of the peak bond stress obtained in B2 compared to the peak value in B1. More uniform bond stress distribution means better and more efficient utilization of the bonded length and higher participation from the rebar ribs closer to the unloaded end. The observation coincides with the expectation of having more slippage of the top reinforcement due to high deformability and low quality of the concrete at the top of the beam as a result of settlement and bleeding effects, as discussed in chapter 2. For the higher stress levels, 100 ksi and 120 ksi, the same trend was also observed, however, at higher bond stress values.

The bond stress distribution shown in figure (5.1) reflects also the effect of the transverse reinforcement which is directly related to the confinement. For beam B3, the transverse reinforcement were # 3 stirrups every 3 in. which is higher than the transverse reinforcement used for beam B1, which were # 3 stirrups every 9 in. It is evident from figure (5.1) that the bond stress distribution in B3 is more uniformly distributed than that in B1. This is evident from the relatively smaller value of the peak bond stress developed in B3 compared to B1. The higher confinement in beam B3 provided better bond stress distribution than for B1. This behavior under high transverse confinement does not match the known behavior of the grade 60 in which the presence of transverse reinforcement will not have
much influence on the bond stress distribution. The influence of the transverse reinforcement in evenly distributing the bond stress and in turn increasing the bond capacity will be again observed in the results of the splice specimens, which will be discussed later. For the higher stress levels, 100 ksi and 120 ksi, the same trend can be seen, however, the peak bond stress values in the two curves increased significantly.

Figure (5.4) presents the bond stress distribution along the bonded length of beam B1 at different stress levels. The bond stress distribution was computed following the above mentioned procedure. From the curve, it could be concluded that for small stress levels, below the proportional limit strength, the location on the bonded length at which the maximum bond stress is developed was at 4 in. from the begging of the bonded length of the bar. However, when the stress in the bar within the bonded length became higher than the proportional limit, the nonlinear behavior of the stress strain curve led to significant increase in the contribution of the ribs near the loaded end due to the tendency of the ribs near the loaded end to excessively elongate. This is evident from the significant increase of the peak bond stress from stress level 100 ksi to 120 ksi.

The applied load and the slip of the unloaded end relationship for the three beams B1, B2, and B3 are shown in figure (5.5). The figures indicate that the slope of the curves for B1 and B3 were very close, however, the slope for B2 was significantly lower. At a load level of about 20 kips, which corresponds to a stress level of about 100 ksi, the three curves started to exhibit a non linear behavior with a significant reduction in slope and the slope of B2 was even more significantly reduced. This nonlinear behavior is due to the non linear response of the MMFX stress-strain curve beyond 100 ksi. Just before reaching the ultimate strength of
the MMFX rebars, an excessive increase in the slip was observed for beams B2 and B3. Figure (5.6) shows the applied load and the slip of the loaded end relationship. A close match was observed between the behavior of the two beams B1 and B2. For beam B3, the slip at any given load level was higher than the slip in B1 and B2 due to the initial slip at the early loading stage. This behavior can be explained in the light of the strain distribution curve for the three beams B1, B2, and B3 at stress level equals 100 ksi, shown in figure (5.7). The strain distribution curve for B3 is higher than other two curves. That implies that the total elongation of the MMFX bar along its bonded length in B3 is more than the elongation in B2, and B1. Since the loaded end slip reflects the amount of bar elongation, it becomes expected to observe higher slippage of the loaded end for B3 than B2 than B1, as shown in figure (5.6).

5.2.3 Evaluation of Test Results Using Descriptive Equations:

Table (5.1) provides the values that describe the level of confinement provided by both concrete and transverse reinforcement surrounding the bonded length for the four tested beams based on the five descriptive equations given in chapter 2. The table, also, shows the limitations that were imposed on the applicability of each equation based on the degree of confinement, as discussed in chapter 2. It is obvious that the levels of confinement provided in beams B1, B2, and B3 were quite higher than the limitations provided in all the five descriptive equations. This implies the ineligibility of using these descriptive equations to predict the bond strength for beams B1, B2, and B3. On the other hand, the level of confinement for beam B4 based on Darwin et al. (1996), Zuo and Darwin (2000), and ACI
committee 408 equations was within the confinement limitation imposed on the three equations. Being within the confinement limits, allow comparing the predictions of these equations to the experimental results. Tables (5.2) and (5.3) present this comparison and from this comparison the following remarks could be concluded.

The closest predicted value to the experimental value regarding the bond force was given by the Zuo and Darwin [2000] equation and the furthest value was given by Darwin et al. [1996]. It is also noticeable that the difference between the predicted value using Zuo and Darwin [2000] equation and that obtained using the ACI committee 408 equation is quite insignificant. It is worth to be mentioned, that the test/predict ratios for these three equations are very close to the average test/predict ratios presented in table (2.2) for conventional carbon steel rebars. Also the test/predict ratios, corresponding to each equation, lie within the standard deviation ranges corresponding to these equations. This observation gives a preliminary idea that there is no indication that the bond strength of the MMFX rebars embedded in concrete is much different from the bond strength of the conventional grade 60 rebars with concrete at 115 ksi stress level.

Despite the fact that the actual level of confinement provided in B1, B2, and B3 exceeded the limitations of the descriptive equations, both the experimental and the predicted bar stress and the experimental and predicted splice lengths and the corresponding test/predict ratios were calculated for B1, B2, B3, and B4 and they are given in table (5.2a) and presented graphically in figures (5.7a) and (5.7b) for the ACI 408 equation and the ACI code 318-02 equation, respectively. From figure (5.7a) it is clear that the ACI 408 equation provided conservative prediction to the bonded length for all beams, however for beam B3,
characterized by having higher levels of confinement, the prediction was enhanced and the ratio get closer to one. For beam B4, as mentioned above, the results showed that the ACI 408 equation could be able to closely predict the splice length of the # 8 MMFX bars. Examining figure (5.7b), the ACI code 318-02 equation provided conservative prediction for all the beams when the confinement level was limited to the 2.5 value given by the code. However, substituting with the actual level of confinement provided in the ACI 318-02 equation will improve the prediction of the equation and get the ratio closer to the unity, except for B3.

It is clear that the ability of both the ACI committee 408 and ACI code 318-02 equations to predict the bond capacity of # 4 MMFX bars becomes more relevant as the level of confinement increases. However, because all the confinement values were beyond the equations’ confinement limitations, it was suggest studying more specimens with confinement levels within the limitations of both the ACI committee 408 and ACI code 318-02 equation. In addition to that, a major difference between the beam end specimens and the splice specimens was observed. The difference is that the test setup used for the former does not allow for beam deflection despite of the section rotation. However, the test setup of the latter allows for deflection. The deflection of the splice specimen will reduce the bond strength of the spliced bars as the deflection simulates the spliced bar to exert additional outward pressure on the bottom concrete cover causing the premature failure of the splice. This idea was supported by the observations made by Ferguson et al. [1962] that increasing the shearing force carried by the dowel action of the reinforcing bar will reduce the bond strength as the shear force in the bar will create additional tensile stresses on the surrounding
cover. Based on that, the second phase of the investigation, in which the splice specimens were used with confinement levels below the limitations of the equations, initiated.

5.3 Test Program II: Splice Specimens

5.3.1 General

The experimental outcomes of the second test program that included testing eight full scale beams reinforced with spliced bars at the mid span will be evaluated. Some observations and remarks will be pointed out from the results.

5.3.2 Comparison between B-6-12 and B-8-12

Beam B-6-12 is reinforced with two # 6 bars and splice length of 12 in. therefore the reinforcement ratio is 0.74 percent. Beam B-8-12 is reinforced with one # 8 bars and splice length of 12 in. therefore the reinforcement ratio is 0.65 percent.

Figure (5.8) represents a comparison made between the mid-span deflections of the two beams throughout the test. There are two important remarks that are worth to be mentioned. Throughout the post cracked portion of the two curves, and at any given load level, the deflection for B-6-12 is always less than that for B-8-12. This behavior is due to the higher stiffness associated with beam B-6-12 as a result of the higher reinforcement ratio relative to beam B-8-12. Throughout the post cracking behavior, the stiffnesses presented by the slopes of the curves were also different. For B-8-12 the slope was about 13.9 kips/in., while for B-6-12 the slope was about 15.5 kips/in. The difference of about 11.5 percent closely matches the 12.5 percent difference in the reinforcement ratio between the two
beams. The ultimate load carrying capacity is also 54 percent higher for beam B-6-12 than B-8-12.

Regarding the comparative load vs. MMFX strain curves, shown in figure (5.9), the same observations, mentioned for mid-span deflection curve, can be made to this relationship. And, again, the higher stiffness of the sections of beam B-6-12 resulting from the difference in cross sectional area between 2 # 6 bars and 1 # 8 bar, will result in a lower bar stress, at any given applied load, compared to beam B-8-12.

5.3.3 Evaluation of Test Results Using Bond Equations:

The relationship between the splice length and both the predicted and the experimental bond force on the left Y-axis and the corresponding bar stress on the right Y-axis for the specimens reinforced with # 6 MMFX bars is shown in figure (5.10). The predicted values for bond force were obtained using the ACI committee 408, Zuo and Darwin [2000], and Darwin et al. (1996b) equations for bond force. From figure (5.10), an observation was made that the three equations conservatively predict the bond force capacity for # 6 MMFX rebars for stress levels in the MMFX bar below the proportional limit, which was found to be 83.5 ksi, and that the predicted values given by these equations progressively become less conservative and eventually turn to be unconservative as the stresses in the MMFX bar exceed the proportional limit strength.

For the short splice lengths and low stress levels in the bar the experimental curve, based on the moment curvature analysis, is almost parallel to the curves of the three predictive equations which were formulated for carbon steel. The similarity in the trend
between the predicted and experimental curves indicted that the behavior of the MMFX steel rebars is closely similar to the behavior of the conventional carbon steel rebars for # 6 bars up to stress level of at least 83.5 ksi, which represent the proportional limit. The ability of these equations to predict the bond force for # 6 MMFX rebars can be numerically examined from the values of test/predict ratios and the percentage difference given in tables (5.4). The deviation of the measured values from the linear relation of the predicted equations is considered to be due to the non-linear behavior of MMFX stress-strain relationship beyond a stress level of 83.5 ksi.

The percentage difference between the predicted and the experimental bond forces for the three above mentioned equations ranged between about 20 percent underestimation, at stress level of 62 ksi, to about 30 percent overestimation, at stress level of 120.6 ksi. Again for the three equations, the test/predict ratios ranged from as minimum as 0.75, at stress level of 62 ksi, to a maximum of about 1.25, at stress level of 120.6 ksi. This implies that the current ACI committee 408 equation can be used to safely predict the bond force capacity of the # 6 MMFX bars with concrete for low stress levels in the bar. However, at higher stress levels the current equation proposed by the ACI committee 408 needs to be modified to avoid any unconservative prediction. Considering the limitation on the maximum stress level that can be considered in design of any type of steel reinforcements, which is imposed by the current ACI code 318-02, which is 80 ksi, the 80 ksi stress level was selected as the upper limit for using the current equation proposed by the ACI committee 408. And for stress levels beyond the 80 ksi, the current equation proposed by the ACI committee 408 will be modified to ensure conservative estimation for bond force up to a stress level of 120 ksi.
In an attempt to modify the ACI committee 408 [2001] equation to enhance its ability to safely predict the bond force capacity of # 6 MMFX rebars not confined with transverse reinforcement for stress level range between 80 ksi and 120 ksi, the following procedure was followed.

The current equation proposed by the current ACI committee 408 [2001] is:

\[
\frac{T_b}{f_c} = \left( 59.9 l_d \left( C_{\text{min}} + 0.5 d_b \right) + 2400 A_b \right) \left( 0.1 \frac{C_{\text{max}}}{C_{\text{min}}} + 0.9 \right)
\]

Equation (5.2)

The philosophy adopted in modify the above equation was based on the idea of preserving the same format of the current equation and changing only the numerical constants. In other words, limiting the changes to be made on the two numerical constants 59.9 and 2400 will save the format of the current equation. Another target was to make sure that all the predictions made by the modified equation will be in the underestimation side of the experimental results and to have both equations, the current and the one after modification, meet at the 80 ksi stress level. This target was achieved by keeping all the test/predict ratios obtained using this modified equation above or equal to the unity. Based on the limited experimental results the values were found to be 34 and 5000 respectively. Therefore the proposed equation could be rewritten in the following format:
Table (5.5) presents both the test/predict ratios and the percentage difference for both the bond force calculated for the bars that developed stress levels beyond 80 ksi, using the above modified equation, and the bond force calculated for the bars that developed stress levels below the 80 ksi limit, using the current ACI 408 equation. Figure (5.11) shows the relationship between the experimental results for the four specimens reinforced with #6 MMFX rebars and the values of the bond force predicted using the current and the modified ACI 408 equation. From the table, it is clear that using the current ACI committee 408 equation, equation (5.2), to predict the bond capacity of #6 MMFX rebars not confined with transverse reinforcement within a stress range between 62 ksi and 80 ksi will guarantee a maximum test/predict ratio of 1.24 and a maximum percentage difference of about 20 percent. And using the modified ACI committee 408 equation, equation (5.3), to predict the bond capacity of #6 MMFX rebars not confined with transverse reinforcement within a stress range between 80 ksi and 120 ksi will guarantee a maximum test/predict ratio of 1.24 and a maximum percentage difference of about 20 percent. From the figure, it can be concluded that the value of the percentage difference between the predicted and the experimental will converge to zero as the stresses in the bar get closer to 120 ksi.

Using the remaining three equations, namely Darwin et al. [1992], ACI 318-02, and OJB [1977] equations to predict the bond capacity of the #6 MMFX bars, the relationship

\[
\frac{T_{b,1/4}}{f_{c}} = \left(34 \ l_{d} \left(C_{\text{min}} + 0.5 \ d_{b}\right) + 5000 \ A_{b} \left(0.1 \ \frac{C_{\text{max}}}{C_{\text{min}}} + 0.9\right) \right)
\]

Equation (5.3)
between the splice length and both the predicted and the experimental bond force is shown in figure (5.12). In figure (5.12), the prediction curves presenting the three above mentioned equations provided safe and conservative prediction to the bond force for the relatively small bar stresses. It was observed that the underestimated predictions for these equations turned rapidly to the overestimation side as the stresses in the bar exceeded the proportional limit strength and went into the non linear zone of the MMFX stress-strain curve. This behavior is more obvious for the two equations of OJB [1977] and ACI-318-02 than for Darwin et al. [1992] equation. The ability of these three equations to predict the bond force for # 6 MMFX rebars can be numerically examined from the values of test/predict ratios and the percentage difference given in tables (5.6).

The percentage difference between the predicted and the experimental bond forces for both Darwin et al. [1992] and OJB [1977] equations ranged between 20 percent underestimation, at the 62 ksi stress level, to about 23 percent overestimation for Darwin et al. [1992] equation and an excessive overestimation of about 47 percent for the OJB [1977] equation at stress level of about 120 ksi. However, the ACI 318-02 equation gave a significant underestimation that reached a maximum value of 35 percent at the 62 stress level and an excessive overestimation that reached 54 percent at the 120 ksi stress level. Again for the three equations, the same trend is observed using the test/predict ratios that ranged from as minimum 0.68, for the 62 ksi stress level, to maximum of about 1.25, for the 120 ksi stress level for Darwin et al. [1992] equation. From the above discussion, it was concluded that the current ACI 318-02 equation can be used to safely predict the bond force capacity of the MMFX bars for relatively low stress levels in the bar. However for higher
stress levels, the ACI 318-02 equation needs to be modified to able to provide safe prediction to the bond force. As in the ACI committee 408 equation, the ACI code 318-02 design stress limit of 80 ksi was selected as the upper boundary stress for using the current ACI 318-02 equation to predict the bond force capacity of the # 6 MMFX bars. And for stress levels beyond the 80 ksi, the current ACI 318-02 equation will to be modified to able to provide safe prediction to the bond force.

In an attempt to modify the current ACI code 318-02 equation so that it could be used to conservatively predict the bond force capacity of # 6 MMFX rebars confined with transverse reinforcement for stress levels in the bar exceeding 80 ksi, the following approach was undertaken.

The equation adopted by the current ACI code 318-02, for bars not confined with transverse reinforcement, in terms of bond force, is in the following format:

\[
\frac{T_b}{f_{Cn/2}} = 3.33l_d \pi \left( C_{min}' + 0.5d_b \right)
\]  

Equation (5.4)

The criterion selected in modify the above equation, was the same criterion used to modify the ACI committee 408 equation; in which we intend to keep the same format of the current ACI 318-02 equation and attempt to change the numerical constants only. In other words, limiting the changes to be made on the numerical constant of the term \(l_d\), which is 3.333, and adding another term function in \(d_b^2\) to the equation. Another target was to make sure that all the test/predict ratios that will be obtained using the equation (5.4), after modification, will be above or equal to the unity to achieve safe prediction for bond force,
and to have both equations, the current and the one after modification, meet at the 80 ksi stress level. Using the limited test results, the selected values were 1.3 and 500 $d_b^2$ as given in equation (5.5):

$$\frac{T_h}{f_c^{3/2}} = 1.3 I_d \pi (C_{min}^' + 0.5d_b) + 500d_b^2$$  \hspace{1cm} \text{Equation (5.5)}

In terms of $L_d$, equation (5.5) will be:

$$\frac{L_d}{d_b} = \frac{f_y}{\sqrt{f_c'}} - \frac{637}{5.2} \left( \frac{C_{min}^' + 0.5d_b}{d_b} \right)$$  \hspace{1cm} \text{Equation (5.6)}

Table (5.7) provides both the test/predict ratios and the percentage difference for both the bond force calculated using equation (5.5) for beams with the bar stress level beyond the 80 ksi and the bond force calculated using equation (5.4) for beams with bar stress level below 80 ksi. Figure (5.13) shows the relationship between the splice lengths and both the experimental results and the predicted values for the four specimens reinforced with # 6 MMFX rebars using both the current and the modified ACI 318-02 equations. From the table, it is clear that using the current ACI 318-02 equation, in terms of bond force, to predict the bond capacity of # 6 MMFX rebars within a stress range between 62 ksi and 80 ksi, will grantees a maximum test/predict ratio of 1.54 and a maximum percentage difference of about
35 percent. And using the modified ACI 318-02 equation, equation (5.5), to predict the bond capacity of #6 MMFX rebars, not confined with transverse reinforcement, within a stress range between 80 ksi and 120 ksi will guarantee a maximum test/predict ratio of 1.2 and a maximum percentage difference of about 16 percent.

The deviation of the prediction curves from the experimental curve could be due to the influence of the non-linear ductile behavior of the MMFX steel rebars at high stress levels on the bond strength. In chapter 2, it was mentioned that according to OJB [1975] the yielding of the conventional carbon steel before bond failure will reduce the bond capacity compared to higher strength bars with the same bonded length and under the same level of confinement but do not yield. The results obtained supports OJB [1975] observation. The results reached by Ferguson and Breen [1965] support the above findings. They tested spliced #8 and #11 bars made of high strength steel having minimum yield strength of 75 ksi. They reported that for bar strain up to 0.006 no signs of reduction in bond strength is observed due to the effect of the shape of the stress-strain diagram. However, for only one beam failed at a bar strain of 0.011 the shape of the stress strain diagram was found to negatively influence the splice strength.

To find the best fitting curve that can match the experimental one, different trend lines were examined and the best to fit was found to be a second degree polynomial equation with a mean square error ($R^2$) equals 0.98. Based on the limited available results, the suggested relationship between splice length and the bond force for bar stress range between 62 ksi and 120 ksi and for #6 MMFX rebars is a 2nd degree relation.

For #8 MMFX rebars the relationship between the splice length and both the
predicted and the experimental bond force is shown in figure (5.14). The predicted values were obtained using the ACI committee 408, Zuo and Darwin [2000], and Darwin et al. [1996b] equations for bond force. From figure (5.14), it is obvious that for the smallest two stress levels, 49 ksi and 73 ksi, corresponding to the shortest two splice lengths, 12 in. and 24 in., the three prediction curves of the ACI committee 408, Zuo and Darwin [2000], and Darwin et al. [1996b] equations give very close prediction to the experimental values. However, for the highest two stress levels, 117 ksi and 138.5 ksi, corresponding to the longest two splice lengths, 48 in. and 72 in., the above equations over estimated the bond force capacity. It was observed from the curve that the degree of over estimation increases gradually as the tensile stresses in the bar exceeded the proportional limit strength which is 88 ksi for # 8 MMFX bars. Table (5.8) provides a numerical evaluation to the relationship shown in figure (5.14) through the test/predict ratios and the percentage difference. As shown in the table, the percentage difference did not exceed 4.3 percent for the three equations when the stress levels in the bars were below 88 ksi which presents the proportional limit. Also, it is worth to be noted that the percentage difference for both equations of ACI committee 408 and Zuo and Darwin [2000] reached about 20 percent over prediction when the level stress in the bar reached 138.5 ksi. From this it could be concluded that the current ACI committee 408 equation for bond force can be used to conservatively predict the bond force capacity of # 8 MMFX bars for relatively low stress levels. However, for higher stress level in the MMFX bar, the current equation proposed by ACI committee 408 needs to be modified to avoid overestimation.

Again, considering the limitation on the maximum stress level that can be considered
in design of any type of steel reinforcements imposed by the current ACI code 318-02, which is 80 ksi, this 80 ksi stress level was selected as the upper limit for using the current equation proposed by the ACI committee 408. And for stress levels beyond the 80 ksi, the current equation proposed by the ACI committee 408 will be modified to ensure conservative estimation for bond force up to a stress level of 138 ksi.

In an attempt to modify the current ACI committee 408 [2001] equation so that it could be used to conservatively predict the bond force capacity of # 8 MMFX rebars confined with transverse reinforcement for bars with stress levels exceeding 80 ksi, the following procedure was taken.

The current equation proposed by the ACI committee 408 [2001] for bars confined with transverse reinforcement is in the following format:

\[
\frac{T_{bh}}{f'_{c}} = \left( 59 \cdot 9 L_{d} \left( C_{\min} + 0.5 d_{h} \right) + 2400 \cdot A_{h} \left( 0.1 \frac{C_{\max}}{C_{\min}} + 0.9 \right) + \left( 30 \cdot 88 t_{s} t_{d} \frac{NA}{n} + 3 \right) \right)^{1/2}
\]

Equation (5.7)

The philosophy selected in modify the above equation, was based on the idea of using the same format of the modified equation, equation (5.3), obtained for the # 6 MMFX rebars not confined by transverse reinforcement and changing only the numerical constants in the added term that presents the effect of the confining transverse reinforcement provided by stirrups. In other words, limiting the changes to be made on the two numerical coefficients 30.88 and (+3) will save the format of the modified equation obtained early. Another target was to make sure that all the test/predict ratios that will be obtained using the equation (5.7)
after modification will be equal to or at least above the unity to ensure safe and underestimated prediction for bond force and to have both equations, the current and the one after modification, meet at the 80 ksi stress level Using the limited test results, the two constants were found to be 35 and (-8) respectively. This change together with the modification made to equation (5.2) will put equation (5.7) in the coming format:

\[
\frac{T_b}{f_c^{1/4}} = \left( 34 \ t_d \left( \frac{C_{\min}}{C_{\min}} + 0.5 \ d_b \right) + 5000 \ A_b \left( 0.1 \ \frac{C_{\max}}{C_{\min}} + 0.9 \right) + \left( 35 \ t_d \ \frac{N_A}{n} - 8 \right) f^{1/2}_c \right) f^{1/2}_c
\]

Equation (5.8)

This equation can be used for predicting bond force for bars with and without confining transverse reinforcement. Table (5.9) provides both the test/predict ratios and the percentage difference for both the bond force calculated using equation (5.8), for MMFX bars with stress levels beyond 80 ksi and the bond force calculated using the current equation (5.7) for MMFX bars that developed stress levels less than the 80 ksi. Figure (5.15) shows the relationship between the predicted and the experimental results for the four specimens reinforced with #8 MMFX rebars. From the table, it is clear that using the current ACI committee 408 equation, equation (5.7), to predict the bond capacity of #8 MMFX rebars confined with transverse reinforcement within a stress range between 49 ksi and 80 ksi, will grantee a minimum test/predict ratio of 0.96, which is slightly unconservative, but a maximum percentage difference of about 3.7 percent. And using the modified equation of the ACI committee 408, equation (5.8), to predict the bond capacity of #8 MMFX rebars confined with transverse reinforcement within a stress range between 80 ksi and 138 ksi will
grantee a maximum test/predict ratio of 1.03 and a maximum percentage difference of about 3.5 percent.

Using the remaining two equations, namely the ACI code 318-02, and OJB [1977] equations to estimate the bond force capacity for the specimens reinforced with # 8 MMFX bars, the relationship between the splice length and both the predicted and the experimental bond force is shown in figure (5.16). In figure (5.16), for # 8 MMFX rebar specimens, the prediction curves presenting the three above mentioned equations provide safe and conservative prediction to the bond force for the relatively low bar stress levels. However, the underestimated predictions for these equations turned rapidly to the overestimation side as the stress developed in the MMFX bar exceeded the proportional limit and the non linear behavior of the MMFX stress-strain curve begin to influence the bond strength. The ability of these equations to predict the bond force for # 8 MMFX rebars can be numerically checked from the values of test/predict ratios and the percentage difference given in tables (5.10).

From table (5.10), it is worth to be mentioned that the percentage difference of underestimation for the ACI 318-02 equation reached a maximum value of about 42 percent when the stress level in the MMFX bar was as low as 49 ksi and around 23 percent overestimation at 138 ksi stress. This implies that the current ACI 318-02 equation, in term of bond force, can be safely used to conservatively predict the bond capacity of # 8 MMFX bars confined with transverse reinforcement at relatively low bar stress levels. However for higher stress levels, the ACI 318-02 equation needs to be modified to able to provide safe prediction to the bond force. As it was mentioned before, the 80 ksi design stress limit imposed by the current code was selected to present an upper boundary for using the current
ACI 318-02 equation to estimate the bond strength of the # 8 MMFX bars. And for stress levels in the bar exceeding 80 ksi, the current ACI 318-02 equation will be modified.

And to modify the current ACI code 318-02 equation so that it could be used to safely predict the bond force capacity of # 8 MMFX rebars confined with transverse reinforcement, the following procedure was followed.

The equation adopted by the current ACI code 318-02, for bars confined with transverse reinforcement, is in the following format:

\[
\frac{T_b}{f_c^{1/2}} = \left( 3.33l_d \pi \left( C_{\min} + 0.5d_h \right) + \frac{\pi l_d A_{tr} f_{yt}}{450sn} \right)
\]

Equation (5.9)

The philosophy adopted in modifying the ACI 318-02 equation, was to keep the same format of the modified ACI 318-02 equation obtained for # 6 bars not confined with transverse reinforcement, equation (5.5), and change only the numerical constant in the term added to the equation that presents the influence of the transverse stirrups, which is equal to 450. Another target was to make sure that all the test/predict ratios that will be obtained after modifying equation (5.9) will be above or equal to the unity to ensure safe and underestimated prediction for bond force and to have both equations, the current and the one after modification, meet at the 80 ksi stress level. In the light of the limited test results, the constant which will replace the 450 is found to be 1130. This changes equation (5.9) to the following format:
In terms of $L_d$ equation (5.10) will be:

$$\frac{T_b}{f_c^{\alpha/2}} = \left( \left[ 1.3l_d \pi (C'_\text{min} + 0.5d_b) + 500d_b \pi d _d + \frac{\pi d A_tr_fy}{1130sn} \right] \right)$$

Equation (5.10)

Table (5.11) provides both the test/predict ratios and the percentage difference for both the bond force calculated using equation (5.9) for MMFX bars that developed stress levels below 80 ksi, and the bond force calculated using equation (5.10) for MMFX bars that developed stress levels above 80 ksi. Figure (5.17) shows the relationship between the predicted and the experimental results for the four specimens reinforced with # 8 MMFX rebars. From the table, it is obvious that using the current ACI 318-02 equation, in terms of bond force, to predict the bond capacity of # 8 MMFX rebars within a stress range between 49 ksi and 80 ksi will grantee a maximum test/predict ratio of 1.72 and a maximum percentage difference of about 42 percent. And using the modified ACI 318-02 equation, equation (5.10), to predict the bond capacity of # 8 MMFX rebars confined with transverse reinforcement within a stress range between 80 ksi and 138 ksi will grantee a maximum test/predict ratio of 1.19 and a maximum percentage difference of about 16 percent.

Table (5.11) provides both the test/predict ratios and the percentage difference for both the bond force calculated using equation (5.9) for MMFX bars that developed stress levels below 80 ksi, and the bond force calculated using equation (5.10) for MMFX bars that developed stress levels above 80 ksi. Figure (5.17) shows the relationship between the predicted and the experimental results for the four specimens reinforced with # 8 MMFX rebars. From the table, it is obvious that using the current ACI 318-02 equation, in terms of bond force, to predict the bond capacity of # 8 MMFX rebars within a stress range between 49 ksi and 80 ksi will grantee a maximum test/predict ratio of 1.72 and a maximum percentage difference of about 42 percent. And using the modified ACI 318-02 equation, equation (5.10), to predict the bond capacity of # 8 MMFX rebars confined with transverse reinforcement within a stress range between 80 ksi and 138 ksi will grantee a maximum test/predict ratio of 1.19 and a maximum percentage difference of about 16 percent.
As in # 6 rebars, to find the best fitting curve that can match the experimental one, different trend lines were tried and the best to fit was, again, found to be a second degree polynomial equation but his time with a mean square error ($R^2$) equals 1.00. That means that based on the limited available results, the suggested relationship between splice length and the bond force for stress level range between 49 ksi and 138 ksi for # 8 MMFX rebars is a 2nd degree relation.

Figure (5.18) shows a comparison between the bond force developed in the # 6 MMFX rebars and the # 8 MMFX rebars at time of bond failure. By comparing the bond force of Beam B-6-12, with a confinement level of 2.6 based on ACI committee 408 equation, and that of beam B-8-12, with a confinement level of 2.8 based also on ACI committee 408 equation, it was found that the ratio between the bond force of B-8-12 to that of B-6-12 is about 1.36 which is slightly higher than the ratio between the perimeters, or the diameters of the two rebars. This implies that the average bond strength for B-8-12 is slightly higher than that for B-6-12. Although the difference in the degree of confinement between the two beams is expected to cause more than a slight change in the bond strength, nevertheless, the increase in the bar size from # 6 to # 8 counteracted such increase in bond strength caused by the increase in confinement level. This behavior is similar to the behavior observed in the conventional carbon steel rebars in which the increase in the level of confinement will lead to an increase in the bond strength; and the increase in the bar size will cause a decrease.

The same comparison mentioned above can be run again between the bond force of beam B-6-24, with a confinement level of 2.6 based on ACI committee 408 equation, and that of beam B-8-24, with a confinement level of 2.9 based also on ACI committee 408
equation. This time the ratio of the bond force of B-8-24 to that of B-6-24 is about 1.48 which is significantly higher than the ratio between the perimeters or the diameters of the two rebars. This implies that the average bond strength for B-8-24 is significantly higher than that for B-6-24. Despite the increase in the bar size but the increase in the level of confinement was higher than the in the case of B-6-12 and B-8-12.

Figure (5.19) presents the relationship between the experimental bond force for # 6 and # 8 MMFX bar specimens and the ratio of the splice length to the bar diameter. From a close examination to the corresponding confinement levels posted on the figure, an observation is made that despite the difference in the level of confinement between # 6 and # 8 specimens, which is about 0.3, an insignificant difference was seen between the two curves for relatively low stress levels below proportional limit. However, at high stress levels, a significant difference was observed. Considering the fact that the # 8 specimens are confined with transverse reinforcement but the # 6 specimens are not, we can conclude that the presence of transverse reinforcement have a remarkable effect on increasing the bond capacity of MMFX at high stress levels within the nonlinear range of the MMFX stress-strain curve. Based on this conclusion, it is recommended to have stirrups as transverse confinement for the splice length when designing the MMFX bars at high stress levels.

The current and the modified equation ACI 408 equation together with the experimental results for # 6 and # 8 MMFX bars are shown in figure (5.20). The current and the modified ACI 318-02 equation together with the experimental results for # 6 and # 8 MMFX bars are shown in figure (5.21).
Figure (5.1) Average Bond Stress Distribution at Stress Level = 80 ksi
Figure (5.2) Average Bond Stress Distribution at Stress Level = 100 ksi
Figure (5.3) Average Bond Stress Distribution at Stress Level = 120 ksi

- B1: #4, bottom, #3 str. @ 9 in.
- B2: #4, top, #3 str. @ 9 in.
- B3: #4, bottom, #3 str. @ 3 in.
Figure (5.4) Bond Stress Distribution along the Bonded Length at different Stress Levels for B1
Figure (5.5) Applied Load vs. Unloaded End Slip for B1, B2, and B3
Figure (5.6) Applied Load vs. Loaded End Slip for B1, B2, and B3
Figure (5.7) MMFX Bar Strain Distributions along the Bonded Length at Stress Level = 100 ksi for B1, B2, and B3.
Figure (5.7a) Test/predict ratio for splice length using ACI committee 408 equation for B1, B2, B3, and B4.
Figure (5.7b) Test/predict ratio for splice length using ACI code 318-02 equation for B1, B2, B3, and B4.
Figure (5.8) Comparing the Applied Load vs. Mid-span Deflection for B-8-12 and B-6-12
Figure (5.9) Comparing the Applied Load vs. MMFX strain for B-6-12 and B-8-12
Figure (5.10) Bond Force and Bar Stress vs. Splice Length for the Beams Reinforced with # 6 MMFX bars using ACI 408, Zuo and Darwin [2000], and Darwin et al. [1996b] equations
Figure (5.11) Bar Stress vs. Splice Length for the Beams Reinforced with # 6 MMFX bars using both Current and modified ACI 408 equations.
Figure (5.12) Bond Force vs. Splice Length for the Beams Reinforced with # 6 MMFX bars using ACI code 318-02, Darwin et al. [1992], and OJB [1977] equations
Figure (5.13) Bar Stress vs. Splice Length for the Beams Reinforced with # 6 MMFX bars using both Current and Modified ACI code 318-02 equations.
Figure (5.14) Bond Force and Bar Stress vs. Splice Length for the Beams Reinforced with # 8 MMFX bars using ACI 408, Zuo and Darwin [2000], and Darwin et al. [1996b] equations
Figure (5.15) Bar Stress vs. Splice Length for the Beams Reinforced with # 8 MMFX bars using both Current and modified ACI 408 equations
Figure (5.16) Bond Force vs. Splice Length for the Beams Reinforced with # 8 MFX bars using ACI code 318-02, and OJB [1977] equations
Figure (5.17) Bar Stress vs. Splice Length for the Beams Reinforced with #8 MMFX bars using both Current and Modified ACI code 318-02 equations
Figure (5.18) Comparison between Bond Force vs. Splice Length for Beams Reinforced with # 6 and # 8 MMFX bars
Figure (5.19) Comparison between MMFX bar Stress vs. \( l_s/d_b \) for Beams Reinforced with # 6 and # 8 MMFX bars
Figure (5.20) Experimental and Predicted Bar Stress vs. Splice Length for # 6 and # 8 MMFX bars using the current and modified ACI committee 408 equation
Figure (5.21) Experimental and Predicted Bar Stress vs. Splice Length for # 6 and # 8 MMFX bars using the current and modified ACI 318-02 equation.
Table (5.1) levels of confinement and confinement limitations for B1, B2, B3, and B4.

<table>
<thead>
<tr>
<th></th>
<th>Level of Confinement</th>
<th>Confinement Limitations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>B1</td>
<td>B2</td>
</tr>
<tr>
<td>OJB (1977)</td>
<td>4.4</td>
<td>4.4</td>
</tr>
<tr>
<td>ACI 318 -02 equ. (12-1)</td>
<td>4.5</td>
<td>4.5</td>
</tr>
<tr>
<td>Darwin et al. (1996b)</td>
<td>5.5</td>
<td>5.5</td>
</tr>
<tr>
<td>Zuo &amp; Darwin (2000)</td>
<td>5.5</td>
<td>5.5</td>
</tr>
<tr>
<td>ACI committee 408</td>
<td>5.5</td>
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</table>

Table (5.2) predicted values for bond force and the test/predict ratios using the descriptive equations for B4*

<table>
<thead>
<tr>
<th></th>
<th>Bond force (lb)</th>
<th>Test/Predict ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>OJB (1977)</td>
<td>107139</td>
<td>1.07</td>
</tr>
<tr>
<td>Darwin et al. (1992)</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Darwin et al. (1996b)</td>
<td>86352.3</td>
<td>1.054</td>
</tr>
<tr>
<td>Zuo &amp; Darwin (2000)</td>
<td>87158.6</td>
<td>1.044</td>
</tr>
<tr>
<td>ACI committee 408</td>
<td>86953.6</td>
<td>1.047</td>
</tr>
<tr>
<td>Experimental</td>
<td>91000.0</td>
<td>1.000</td>
</tr>
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</table>
### Table (5.2a) Experimental and predicted bar stress and splice length and corresponding test/predict ratios and level of confinement

<table>
<thead>
<tr>
<th>For Beam</th>
<th>Using</th>
<th>Bar Stress (psi)</th>
<th>Test/predict ratio</th>
<th>Is (in.)</th>
<th>Is Test/Is Predict ratio</th>
<th>confinement</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
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<tr>
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<td>ACI 318 -02</td>
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</tr>
<tr>
<td></td>
<td>Darwin et al. (1996b)</td>
<td>132422</td>
<td>1.30</td>
<td>22.82</td>
<td>0.61</td>
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</tr>
<tr>
<td></td>
<td>Zuo &amp; Darwin (2000)</td>
<td>139537</td>
<td>1.23</td>
<td>20.34</td>
<td>0.69</td>
<td>5.5</td>
</tr>
<tr>
<td></td>
<td>ACI 408</td>
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<td>1.25</td>
<td>20.22</td>
<td>0.69</td>
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</tr>
<tr>
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<td>15.18</td>
<td>0.92</td>
<td>4.5</td>
</tr>
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<td>Zuo &amp; Darwin (2000)</td>
<td>139537</td>
<td>1.18</td>
<td>19.37</td>
<td>0.72</td>
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<td>13.79</td>
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<tr>
<td></td>
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<td>1.31</td>
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<tr>
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<td>Darwin et al. (1996b)</td>
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<td>0.97</td>
<td>7.0</td>
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<td>Zuo &amp; Darwin (2000)</td>
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<td>0.99</td>
<td>15.23</td>
<td>0.92</td>
<td>7.0</td>
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<tr>
<td></td>
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<td>0.93</td>
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<td>14.00</td>
<td>1.00</td>
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<td>OJB (1977)</td>
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<td>37.37</td>
<td>0.91</td>
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<td>Darwin et al. (1996b)</td>
<td>109582</td>
<td>1.05</td>
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<td>0.94</td>
<td>3.8</td>
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<tr>
<td></td>
<td>Zuo &amp; Darwin (2000)</td>
<td>110507</td>
<td>1.04</td>
<td>37.11</td>
<td>0.92</td>
<td>3.8</td>
</tr>
<tr>
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<td>36.82</td>
<td>0.92</td>
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<td>Experimental</td>
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Table (5.3) predicted values for bond force and test/predict ratios using design equations for B4

<table>
<thead>
<tr>
<th>Bond force (lb)</th>
<th>Test/Predict ratio</th>
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</thead>
<tbody>
<tr>
<td>ACI 318 -02 equ. (12-1)</td>
<td>83406.3</td>
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<tr>
<td>ACI committee 408 with $\Phi = 0.92$</td>
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</tr>
<tr>
<td>ACI committee 408 with $\Phi = 0.82$</td>
<td>73285.4</td>
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<tr>
<td>Experimental</td>
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</tbody>
</table>

* the experimental bond force was computed using the axial tensile stress, calculated using the moment curvature analysis, and the bar nominal diameter.

Table (5.4) test/predict ratio and percentage difference based on the ACI committee 408, Zuo and Darwin [2000] and Darwin et al. [1996] for the specimens reinforced # 6 MMFX bar.

<table>
<thead>
<tr>
<th>Test/predict ratio</th>
<th>% difference*</th>
<th>MMFX rebar stress (ksi)</th>
<th>Splice length (in.)</th>
<th>Beam ID</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI committee 408</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.24</td>
<td>19.5</td>
<td>62.0</td>
<td>12</td>
<td>B-6-12</td>
</tr>
<tr>
<td>1.14</td>
<td>12.3</td>
<td>87.8</td>
<td>24</td>
<td>B-6-24</td>
</tr>
<tr>
<td>1.12</td>
<td>10.4</td>
<td>115.0</td>
<td>36</td>
<td>B-6-36</td>
</tr>
<tr>
<td>0.78</td>
<td>-27.7</td>
<td>120.6</td>
<td>60</td>
<td>B-6-60</td>
</tr>
<tr>
<td>Zuo &amp; Darwin (2000)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.25</td>
<td>20.3</td>
<td>62.0</td>
<td>12</td>
<td>B-6-12</td>
</tr>
<tr>
<td>1.15</td>
<td>12.9</td>
<td>87.8</td>
<td>24</td>
<td>B-6-24</td>
</tr>
<tr>
<td>1.12</td>
<td>10.9</td>
<td>115.0</td>
<td>36</td>
<td>B-6-36</td>
</tr>
<tr>
<td>0.79</td>
<td>-27.1</td>
<td>120.6</td>
<td>60</td>
<td>B-6-60</td>
</tr>
<tr>
<td>Darwin et al. (1996b)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.27</td>
<td>21.1</td>
<td>62.0</td>
<td>12</td>
<td>B-6-12</td>
</tr>
<tr>
<td>1.13</td>
<td>11.8</td>
<td>87.8</td>
<td>24</td>
<td>B-6-24</td>
</tr>
<tr>
<td>1.10</td>
<td>9.0</td>
<td>115.0</td>
<td>36</td>
<td>B-6-36</td>
</tr>
<tr>
<td>0.76</td>
<td>-31.2</td>
<td>120.6</td>
<td>60</td>
<td>B-6-60</td>
</tr>
</tbody>
</table>
Table (5.5) test/predict ratios and percentage difference for bond force obtained using both the current and the modified ACI committee 408 equation for the specimens reinforced with # 6 MMFX bars.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Test/predict ratio</th>
<th>% difference*</th>
<th>Using the</th>
<th>MMFX bar stress level (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-6-12</td>
<td>1.24</td>
<td>19.5</td>
<td>current equation</td>
<td>62.0</td>
</tr>
<tr>
<td>B-6-24</td>
<td>1.14</td>
<td>12.3</td>
<td>current equation</td>
<td>87.8</td>
</tr>
<tr>
<td>B-6-36</td>
<td>1.24</td>
<td>19.2</td>
<td>modified equation</td>
<td>115.0</td>
</tr>
<tr>
<td>B-6-60</td>
<td>1.00</td>
<td>0.98</td>
<td>modified equation</td>
<td>120.6</td>
</tr>
</tbody>
</table>

* % difference = (Experimental-Predicted)/Experimental x100

Table (5.6) test/predict ratio and percentage difference based on the ACI 318-02, Darwin et al. [1992] and OJB [1977] for the specimens reinforced # 6 MMFX bar.

<table>
<thead>
<tr>
<th>Test/predict ratio</th>
<th>% difference*</th>
<th>MMFX rebar stress (ksi)</th>
<th>Splice length (in.)</th>
<th>Beam ID</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.54</td>
<td>35.0</td>
<td>62.0</td>
<td>12</td>
<td>B-6-12</td>
</tr>
<tr>
<td>1.09</td>
<td>8.1</td>
<td>87.8</td>
<td>24</td>
<td>B-6-24</td>
</tr>
<tr>
<td>0.96</td>
<td>-3.6</td>
<td>115.0</td>
<td>36</td>
<td>B-6-36</td>
</tr>
<tr>
<td>0.64</td>
<td>-56.1</td>
<td>120.6</td>
<td>60</td>
<td>B-6-60</td>
</tr>
</tbody>
</table>

ACI 318-02

| 1.25               | 20.1          | 62.0                    | 12                  | B-6-12    |
| 1.13               | 11.8          | 87.8                    | 24                  | B-6-24    |
| 1.10               | 9.1           | 115.0                   | 36                  | B-6-36    |
| 0.81               | -23.4         | 120.6                   | 60                  | B-6-60    |

Darwin et al [1992]

| 1.24               | 19.5          | 62.0                    | 12                  | B-6-12    |
| 1.03               | 3.4           | 87.8                    | 24                  | B-6-24    |
| 0.97               | -3.2          | 115.0                   | 36                  | B-6-36    |
| 0.68               | -47.0         | 120.6                   | 60                  | B-6-60    |

OJB [1977]
Table (5.7) test/predict ratios and percentage difference for bond force calculated using both the current and the modified ACI code 318-02 equation for the specimens reinforced with # 6 MMFX bars.

<table>
<thead>
<tr>
<th>Specimens ID</th>
<th>Test/predict ratio</th>
<th>% difference*</th>
<th>Using the</th>
<th>MMFX bar stress level (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-6-12</td>
<td>1.54</td>
<td>35.0</td>
<td>current equation</td>
<td>62.0</td>
</tr>
<tr>
<td>B-6-24</td>
<td>1.04</td>
<td>9.0</td>
<td>modified equation</td>
<td>87.8</td>
</tr>
<tr>
<td>B-6-36</td>
<td>1.19</td>
<td>15.88</td>
<td>modified equation</td>
<td>115.0</td>
</tr>
<tr>
<td>B-6-60</td>
<td>1.01</td>
<td>0.89</td>
<td>modified equation</td>
<td>120.6</td>
</tr>
</tbody>
</table>

* % difference = (Experimental-Predicted)/Experimental x100

Table (5.8) test/predict ratio and percentage difference based on the ACI 408 committee, Zuo and Darwin [2000] and Darwin et al. [1996] for the specimens reinforced # 8 MMFX bar.

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Beam ID</th>
<th>Beam ID</th>
<th>Beam ID</th>
<th>Beam ID</th>
<th>Beam ID</th>
<th>Beam ID</th>
<th>Beam ID</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test/predict ratio</td>
<td>% difference*</td>
<td>MMFX rebar stress (ksi)</td>
<td>Splice length (in.)</td>
<td>Beam ID</td>
<td>Test/predict ratio</td>
<td>% difference*</td>
<td>MMFX rebar stress (ksi)</td>
</tr>
<tr>
<td>ACI committee 408</td>
<td>ACI committee 408</td>
<td>ACI committee 408</td>
<td>ACI committee 408</td>
<td>ACI committee 408</td>
<td>ACI committee 408</td>
<td>ACI committee 408</td>
<td>ACI committee 408</td>
</tr>
<tr>
<td>0.96</td>
<td>-3.7</td>
<td>49.0</td>
<td>12</td>
<td>B-8-12</td>
<td>0.96</td>
<td>-3.7</td>
<td>49.0</td>
</tr>
<tr>
<td>0.99</td>
<td>-0.9</td>
<td>73.0</td>
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<td>B-8-24</td>
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<td>-6.7</td>
<td>117.0</td>
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<tr>
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<td>-19.0</td>
<td>138.5</td>
<td>72</td>
<td>B-8-72</td>
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<td>-3.7</td>
<td>117.0</td>
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<tr>
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<td>-4.3</td>
<td>49.0</td>
<td>12</td>
<td>B-8-12</td>
<td>0.96</td>
<td>-4.3</td>
<td>49.0</td>
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<tr>
<td>0.99</td>
<td>-1.3</td>
<td>73.0</td>
<td>24</td>
<td>B-8-24</td>
<td>0.93</td>
<td>-7.0</td>
<td>117.0</td>
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<tr>
<td>0.84</td>
<td>-19.3</td>
<td>138.5</td>
<td>72</td>
<td>B-8-72</td>
<td>0.84</td>
<td>-19.3</td>
<td>138.5</td>
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<tr>
<td>1.04</td>
<td>4.1</td>
<td>49.0</td>
<td>12</td>
<td>B-8-12</td>
<td>0.96</td>
<td>-3.7</td>
<td>117.0</td>
</tr>
<tr>
<td>1.03</td>
<td>3.1</td>
<td>73.0</td>
<td>24</td>
<td>B-8-24</td>
<td>0.84</td>
<td>-18.7</td>
<td>138.5</td>
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</tbody>
</table>

* % difference = (Experimental-Predicted )/Experimental x100
Table (5.9) test/predict ratios and percentage difference for bond force obtained using both the current and the modified ACI committee 408 equation for the specimens reinforced with # 8 MMFX bars.

<table>
<thead>
<tr>
<th>Specimens ID</th>
<th>Test/predict ratio</th>
<th>% difference*</th>
<th>Using the</th>
<th>MMFX bar stress level (ksi)</th>
</tr>
</thead>
<tbody>
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</tr>
<tr>
<td>B-8-24</td>
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<td>-0.90</td>
<td>current equation</td>
<td>73.0</td>
</tr>
<tr>
<td>B-8-48</td>
<td>1.03</td>
<td>3.46</td>
<td>modified equation</td>
<td>117.0</td>
</tr>
<tr>
<td>B-8-72</td>
<td>1.00</td>
<td>0.04</td>
<td>modified equation</td>
<td>138.5</td>
</tr>
</tbody>
</table>

* % difference = (Experimental-Predicted)/Experimental x100

Table (5.10) test/predict ratio and percentage difference based on the ACI 318-02, and OJB [1977] for the specimens reinforced # 8 MMFX bar.

<table>
<thead>
<tr>
<th>Test/predict ratio</th>
<th>% difference*</th>
<th>MMFX rebar stress (ksi)</th>
<th>Splice length (in.)</th>
<th>Beam ID</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>ACI 318-02</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.72</td>
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<td>49.0</td>
<td>12</td>
<td>B-8-12</td>
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<tr>
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<td>21.3</td>
<td>73.0</td>
<td>24</td>
<td>B-8-24</td>
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<tr>
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<td>48</td>
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<td>0.81</td>
<td>-23.2</td>
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<td>72</td>
<td>B-8-72</td>
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<td><strong>OJB [1977]</strong></td>
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<td>1.23</td>
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<td>-17.2</td>
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</table>

* % difference = (Experimental-Predicted)/Experimental x100
Table (5.11) test/predict ratios and percentage difference for bond force calculated using both the current and the modified ACI code 318-02 equation for the specimens reinforced with # 8 MMFX bars

<table>
<thead>
<tr>
<th>Specimens ID</th>
<th>Test/predict ratio</th>
<th>% difference*</th>
<th>Using the</th>
<th>MMFX bar stress level (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-8-12</td>
<td>1.72</td>
<td>41.8</td>
<td>current equation</td>
<td>49.0</td>
</tr>
<tr>
<td>B-8-24</td>
<td>1.27</td>
<td>21.3</td>
<td>current equation</td>
<td>73.0</td>
</tr>
<tr>
<td>B-8-48</td>
<td>1.15</td>
<td>13.4</td>
<td>modified equation</td>
<td>117.0</td>
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<td>B-8-72</td>
<td>1.19</td>
<td>16.3</td>
<td>modified equation</td>
<td>138.5</td>
</tr>
</tbody>
</table>

* % difference = (Experimental-Predicted)/Experimental x100
CHAPTER 6  SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 General

In this chapter a brief summary of the experimental investigation is presented. This will include summarizing the research significance, objectives, scope and findings. Based on the overall evaluation of each parameter considered in this investigation several conclusions are provided. At the end of the chapter, recommendations regarding potential future work in this field are suggested based on the obtained results.

6.2 Summary

To achieve confident and efficient utilization of a new type of steel rebars, reliable and comprehensive information about its mechanical properties of concern and its composite behavior with concrete should be well defined. Bond characteristics between concrete and reinforcing bars are one of the major structural properties that influence the strength and the performance of flexural concrete members. The objective of the research work presented in this thesis is to determine experimentally the bond behavior of straight deformed MMFX steel rebars embedded in concrete flexural members. The research included evaluation of the ability of the current expressions to predict the bond strength of the MMFX rebars with concrete. In this regard two phases of experimental investigations were conducted. In phase I, four beam end specimens each reinforced in tension with one MMFX rebar were tested. In phase II, eight fully scaled spliced beam specimens were examined. For the beam end specimens, the slips at both the loaded end
and the unloaded ends were measured and the strain distribution along the bonded length was monitored. For the splice specimens, mid span deflection, concrete compression strain and MMFX rebar strain were recorded. Close similarity in the bond behavior of the MMFX and the carbon steel was observed. At higher stress levels, beyond the proportional limit, the bond failure changed from the typical sudden and brittle failure, normally observed for conventional steel, to a gradual and ductile failure especially in the presence of transverse reinforcements around the spliced length. The equation proposed by the current ACI committee 408 for bond force was able to provide very closely prediction to the bond force capacity, and consequently the splice length, for # 8 MMFX rebars at low tensile stress levels in the bar and the maximum percentage difference was only 4 percent at stress level of 49 ksi. However, for # 6 bars, the equation proposed by the current ACI committee 408 provided an underestimation to the bond force capacity at low stress levels in the bar within the linear portion of the stress-strain relationship curve and the maximum percentage difference reached about 19 percent at stress level of 62 ksi. On the other hand, the ACI committee 408 equation for bond force gradually overestimated the bond force capacity for both # 6 and # 8 MMFX rebars when the stress levels in the MMFX bar exceeded the proportional limit, and this overestimation percentage reached a maximum value of 19 percent for # 8 bars at stress level of about 138 ksi and reached 28 percent for # 6 bars at stress level of 120 ksi. The ACI 318-02 code equation, in terms of bond force, gave an excessively under estimated prediction to the bond force capacity for both # 6 and # 8 MMFX rebars at low tensile stress levels in the bar. The degree of underestimation increased as the bar stress decreased, and the
percentage difference reached a maximum value of about 42 percent at stress level of 49 ksi for # 8 bars, and reached a maximum value of 35 percent at stress level of 62 ksi for # 6 bars. The ACI 318-02 code equation, in terms of bond force, progressively overestimated the bond force capacity for both # 6 and # 8 MMFX rebars when the tensile stress levels developed in the MMFX bar exceeded the proportional limit. The degree of overestimation for # 8 bars reached a maximum value of 23 percent at stress level of about 138 ksi, and reached 56 percent for # 6 bars at stress level 120 ksi. The 80 ksi design stress limit imposed by the current code ACI 318-02 was selected as an upper boundary for using the both the current ACI committee 408 and the current ACI 318-02 code equations for bond force to predict the bond capacity of the # 6 and # 8 MMFX bars. For stress levels exceeding 80 ksi and up to a stress level of 138 ksi for # 8 bars and 120 ksi for # 6 MMFX bars, both the ACI committee 408 equation and the ACI 318-02 code equation were modified to avoid any overestimated prediction to the bond force of the MMFX rebars. In the process of the modification, the original format of the equation was preserved and only the numerical constants were adjusted. A second degree relationship between bond force and the splice length is suggested to be the best to define the MMFX rebar bond behavior for a stress range between 49 ksi and 138 ksi for # 8 bars and between 62 ksi and 120 ksi for # 6 MMFX bars with a square errors of $R^2 = 0.99$ for # 8 bars and $R^2 = 0.98$ for # 6.

### 6.3 Conclusions

The following conclusions were made based on the observations and the findings
obtained from the limited number of tested specimens available.

1. The investigation results showed that there is no reason to believe that the bond behavior of the MMFX rebars is different than that of the conventional carbon steel for stress levels below the proportional limit of the MMFX steel.

2. The nonlinear behavior of the MMFX bars at high stress levels is considered to be the reason behind the change in the mode of failure from sudden to gradual.

3. Test results indicate that the nonlinear ductile response of the MMFX rebars at high stress levels beyond proportional limit strength, has a strong influence in reducing the bond strength of the MMFX rebars compared to the bond strength that can be obtained when using any other type of steel bars with the same splice length and level of confinement, but with linear stress-strain relation at high stress levels.

4. The current equation proposed by the ACI committee 408, for bond force, for bars confined with and without transverse reinforcement can be used to conservatively predict the bond capacity of the # 6 and # 8 MMFX rebars for relatively low stress level in the bar.

5. For higher stress levels, the current equation proposed by the ACI committee 408
needs to be adjusted to provide safe prediction to the bond force capacity of both 
# 6 and # 8 MMFX bars either confined or not confined with transverse reinforcement.

6. The ACI 318-02 current equation, in terms of bond force, can be used to 
conservatively predict the bond capacity of beams reinforced with # 6 and # 8 
MMFX rebars for relatively low stress levels.

7. For higher stress levels, the current ACI 318-02 equation needs to be modified to 
provide conservative prediction to the bond force capacity of beams reinforced 
with # 6 and # 8 MMFX bars.

8. Confining the splice length with transverse reinforcement will cause significant 
enhancement to the bond force capacity for beams reinforced with MMFX bars 
when the stress level in the MMFX bar exceeds the proportional limit.

6.4 Recommendations

Based on the mechanical properties investigation that included studying the 
tensile strength properties of the MMFX bars, mentioned in chapter 2, the yield strength 
of the MMFX steel rebars, based on the 0.2 percent offset method, is in the range of 118 
ksi to 120 ksi and the proportional limit strength was in the range of 83 ksi to 88 ksi. That 
implies that the nonlinear zone of the stress strain curve between a stress level of 83 ksi
and 118 ksi for # 6 and between a stress level of 88 ksi and 120 ksi for # 8 bars is expected to be part of the tensile response that will develop in the MMFX rebars before reaching the yielding strength. This fact emphasizes the importance of studying the bond behavior of the MMFX rebars within the post linear portion of the stress-strain curve. The currently available equations for bond force are unable to safely predict the bond capacity of the MMFX rebars when the stress levels in the bars exceed the linear limit, the need for a new equation that best describes the bond strength of the MMFX rebars arises. To reach such realistic equation the following recommendations are suggested:

1. More splice beam specimens with different practical levels of confinement, with and without transverse reinforcement, and for wider range of stress levels need to be tested to obtain more detailed information about the bond behavior of the MMFX bars.

2. Different bar sizes have to be considered in the future work, in addition to # 6 and # 8, to examine the influence of the bar size on the bond properties of the MMFX bars.

3. Concrete beams fully reinforced with MMFX rebars in the tension zone, in the compression zone, and in the shear zone, as transverse stirrups, need to be studied.
4. More specimens are needed to thoroughly study the influence of the confining transverse reinforcement in enhancing the bond capacity of the MMFX bars developing stresses beyond the proportional limit.

5. Finding the numerical coefficients and the variables of the proposed 2nd degree relation believed to be the best to describe the relationship between the splice length and the bond force for both # 6 and # 8 MMFX bars.

6. Conducting an analytical investigation to study the bond properties of the MMFX steel rebars with concrete using the actual material properties and following the same scheme that was followed in studying the bond characteristics of grade 60 steel is expected to save some time and money.
1. A 615/A 615M Standards Specifications for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement.


4. ACI Committee 318, 2002, *Building Code requirements for structural Concrete (318-02) and Commentary (318R-02)*, American Concrete Institute, Farmington Hills, MI.


27. MMFX product Bulletin [2002].


