ABSTRACT

MUNIKRISHNA, ARUNA. Shear Behavior of Concrete Beams Reinforced with High Performance Steel Shear Reinforcement. (Under the direction of Dr. Sami Rizkalla.)

The current shear design provisions of the ACI 318-05 specifications limit the yield strength in transverse reinforcement to 60 ksi. Advancement in technology has led to the development of high performance (HP) steel. Use of HP steel in reinforced concrete could lead to cost savings by reducing the amount of steel required due to its high strength and increase in the service life of structural members due to its enhanced corrosion resistance. Commercially available steel, Micro-Composite Multi-Structural Formable (MMFX), conforming to ASTM A 1035, was selected for this study. MMFX steel has specified minimum yield strength of 100 ksi.

This experimental program comprised eighteen tests using nine large-sized reinforced concrete beams subjected to static loading up to failure. The key parameters considered in the experimental program were steel type and the amount of shear reinforcement. This research investigated crack width, modes of failure, deflection, stirrup strain, ultimate load carrying capacity and the behavior of the MMFX steel as transverse reinforcement for concrete beams.

Results from the experimental program showed that by utilizing the higher yield strength and consequently reducing the reinforcement ratio of MMFX steel, the beams can achieve almost the same load carrying capacity as the beams reinforced with conventional Grade 60 steel. Also, beams reinforced with MMFX showed similar deflections at service load as the beams reinforced with Grade 60 steel. Therefore, reduction in the reinforcement ratio of MMFX steel, did not affect the serviceability of these beams.
Analysis shows that the ACI 318-05, CSA A23.3-04, and AASHTO LRFD-04 design codes can closely predict the ultimate shear strength for beams reinforced with high performance steel having yield strength up to 100 ksi. The beams were also analyzed using a well established Modified Compression Field Theory (MCFT) to predict the shear strength of beams reinforced with high performance steel. MCFT can be used to accurately predict the shear behavior of the beams.

Based on the results and findings of the experimental and analytical research, design guidelines are proposed for the use of MMFX steel as shear reinforcement in concrete beams.
Shear Behavior of Concrete Beams Reinforced with High Performance Steel Shear Reinforcement

by

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DEDICATION

To my wonderful parents Nalini and Munikrishna, who have raised me to be the person I am today. They taught us, their children, the value of education and made sacrifices so that we could have the opportunities they did not have.

To my brother Harish Munikrishna, who always believed in me and offered unconditional love at every phase of my life.

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BIOGRAPHY

Aruna Munikrishna was born to Munikrishna and Nalini on 11th July, 1983 in Bangalore, India. She started her engineering in October 2001 at R.V.College of Engineering, Bangalore, India. In 2005, she graduated with distinction and received her Bachelor of Engineering (B.E) degree in Civil. Upon graduation she was appointed as a Junior Design Engineer with China State Construction Engineering (HK) Ltd. She joined North Carolina State University, Raleigh in Fall 2006 to pursue Master of Science degree in Civil Engineering, with a concentration in Structures and Mechanics. She has been working under the guidance of Dr. Sami Rizkalla as a part of the Constructed Facilities Laboratory at North Caroline State University. Upon completion of her Master’s, she will begin working for Jacobs Engineering, Raleigh, NC.
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1. INTRODUCTION

Corrosion of steel reinforcement is one of the major causes of deterioration of reinforced concrete facing the civil engineering infrastructure. Corrosion causes expansion of the overall volume of reinforcing bars, this expansion causes internal pressures to develop in concrete, and often leads to cracking, spalling, and deterioration of concrete structures. Advances in material science have led to the production of High Performance reinforcing bars as an alternative reinforcement for concrete structures. High performance steel is characterized by its high tensile strength and enhanced corrosion resistance in comparison to conventional ASTM A615 Grade 60 steel. Typical steel reinforcing bars currently in use for concrete structures are designed for yield strength of 60 ksi. Use of high performance steel offers several advantages such as reduction of the reinforcement ratio, smaller cross sections which lead to reduction of dead loads, and possible longer spans. The amount of reinforcement required could be considerably reduced by utilizing the high tensile strength. This would eliminate the reinforcement congestion and improve the overall concrete placement. In addition, the use of high strength and high performance steel leads to reduced construction costs and possible increase of service life due to enhanced corrosion resistance.

The new commercially available, Micro-composite Multi-structural Formable (MMFX) steel provides a high level of corrosion resistance and higher tensile strength in comparison to conventional Grade 60 steel. The MMFX steel technology was developed by Professor Gareth Thomas at the National Center for Electron Microscopy at the Lawrence Berkeley National Laboratory. The high strength and high performance was achieved by the
alteration of the microstructure during the production process. The proprietary material composition and production processes minimize the formation of micro galvanic cells and delay the initiation of corrosion. The advantages of higher strength and corrosion resistance do not come without tradeoffs. The consequence of a retooled microstructure is a stress-strain curve that varies considerably from that of conventional Grade 60 steel. The elastic-plastic behavior of MMFX steel does not have the advantage of a yielding plateau which increases the overall ductility of reinforced concrete structures. Because of the non-linearity of stress-strain curve, the shear behavior of beams reinforced with high performance steel has to be investigated. Test results by the MMFX Technologies Corporation (2005) have shown that MMFX rebars could achieve minimum yield strength of 100 ksi using the 0.2% offset method. One major concern with steel having yields in excess of 80 ksi is that the high induced strains would create large cracks in reinforced concrete. This problem leads to potential concern for the application of MMFX in reinforced concrete structures.

To address this problem, an investigation on this topic was carried out at the Constructed Facilities Laboratory (CFL) in North Carolina State University, Raleigh to provide a better understanding of the MMFX reinforcing steel to the construction industry. In this project, eighteen shear tests were conducted using nine large sized reinforced concrete beams. Based on this research and previous research conducted at CFL, provisions to support the design of stirrups with yield strength of 100 ksi, without impairing the ultimate load carrying capacity or altering the serviceability behavior was developed.
1.1 Significance

Currently, there are no experimental data or design guidelines for the use high performance MMFX steel as shear reinforcement with yield strength of 100 ksi in reinforced concrete structures. Most of the research currently available in the literature focused on the use of MMFX as flexural reinforcement and direct replacement of conventional Grade 60 stirrups with MMFX stirrups for concrete structures. The research findings of this study will provide fundamental understanding of the behavior of MMFX stirrups designed for yield strength of 80 ksi and 100 ksi in reinforced concrete structures. This research includes experimental and analytical phases to understand the behavior. The research will provide design guidelines for practioners to use MMFX as shear reinforcement in flexure members.

1.2 Objectives

The overall objective of this report is to understand the shear behavior of flexural members reinforced with MMFX as shear reinforcement. In order to achieve this objective, the following specific research activities were undertaken.

1. Examine the concrete shear resistance, $V_c$, contribution of flexural member reinforced with longitudinal MMFX steel and without shear reinforcement.
2. Determine the total nominal shear strength of beams reinforced with MMFX longitudinal bars and MMFX stirrups.
3. Examine the applicability of building codes for predicting the resistance of flexural members reinforced with high strength shear reinforcement.
4. Evaluate the capability of current computer aid like RESPONSE 2000 in predicting the behavior of flexural member reinforced with MMFX as shear reinforcement.

5. Provide design recommendations on the use of MMFX as shear reinforcement for flexural members.

1.3 Scope

In order to achieve the objectives mentioned above, nine large scale beams were constructed and tested at the Constructed Facilities Laboratory, at North Carolina State University. All beams were tested to failure under monotonic loading. The test results were analyzed to evaluate the shear behavior of MMFX steel.

In addition to this introductory chapter, this report consists of four chapters.

Chapter 2 reviews the relevant research conducted on parameters affecting the shear resistance of concrete, shear models typically used to evaluate shear behavior and also presents shear provisions in current design codes.

Chapter 3 describes the design of test specimens, the experimental program used to determine material properties, the fabrication of specimens, test setup, instrumentation and test procedure.

Chapter 4 presents the test results and analysis obtained from the experimental program, examines the observed modes of failure and evaluates the performance of current shear provisions.
Chapter 5 summarizes the conclusions findings of this research and presents recommendations for future research.
2. LITERATURE REVIEW

This chapter provides a brief review of the current knowledge on the shear behavior of reinforced concrete beams. This review will underline the major parameters that have significant influence on shear behavior including the several different models which have been proposed to determine the shear capacity of reinforced concrete members. The literature review includes also the rational models which have been proposed by others to describe the shear behavior. Some of these have been implemented in codes of practice. In addition, the chapter reviews briefly the current code provisions for predicting the shear capacity of reinforced concrete beams are also included. The basic characteristics of MMFX rebars are highlighted in this chapter. Also, a brief review of use of MMFX as shear reinforcement is included in this chapter.

2.1 Shear in concrete beams

Shear failure is characterized by the formation of a single diagonal or a series of diagonal cracks occurring at an angle with respect to the beam axis. The diagonal cracks occur due to presence of diagonal tension in the reinforced concrete beam hence, shear failure is also known as diagonal tension failure. As shear failures are sudden and brittle, it is recommended that the shear strength of a beam must exceed its flexural strength. Shear failure in reinforced concrete has received lot of attention due to the complexity of shear resistance mechanism. Over the years, intensive research has been carried out through out the
world to provide analytical shear design models. The difficulty in establishing a completely analytical shear design model lies in the fact that shear failure is the sum of several mechanisms with discontinuity and rigid body motion therefore can not be simplified similar to flexural behavior where the plane section remain plane after deformation. The internal forces in a reinforced concrete beam with diagonal crack are shown in Figure 2-1.

![Internal Forces Diagram](image)

**Figure 2-1: Internal Forces (Nilson et al. 2004)**

C is the compressive force in uncracked concrete

\( V_{cz} \) is the shear in the compression zone

\( V_s = A_s f_s \) is the shear transferred by tension in stirrups

\( V_i \) is the shear transferred by the aggregate interlock

\( T = A_s f_s \) is the Tension in longitudinal steel

\( V_d \) is the dowel action of longitudinal reinforcement

s is the stirrup spacing
Compressive force $C$, in the uncracked concrete can resist applied shear forces even after the formation of diagonal crack. Before the first diagonal crack occurs, the applied shear is resisted by the internal friction along the crack due to surface roughness of the concrete, $V_i$, the vertical forces across the longitudinal steel, $V_d$ and the shear in the compression zone, $V_{cz}$. As it is difficult to quantify the degree of contribution of these factors individually, researchers have collectively grouped these factors as the concrete contribution $V_c$ (MacGregor et al., 2005). Stirrups contribute in resisting the applied shear only after the formation of the first diagonal crack. Collins (1996) demonstrated that cracked concrete possesses tensile stresses that can considerably increase the ability of concrete to resist shear forces.

2.2 Shear Strength parameters

The important factors affecting the shear strength of reinforced concrete beams are discussed in the following sections:

Concrete Strength:

The effect of concrete compressive strength on the shear resistance has been studied by many researchers. In general, the contribution of concrete to shear resistance typical considered to be $2\sqrt{f'_c}bd$ (ACI 318 – 05). This simple equation was developed by Moody et al. in 1954. According to the equation, as the concrete strength is increased the shear strength also increases. Shear cracks in beams built with high strength concrete are smoother than
cracks in beams built with normal strength concrete. This is because the cracks in high strength concrete pass through the coarse aggregate and in normal strength concrete the cracks pass around the aggregates. With smoother cracks the internal friction generated due to the surface roughness of the crack will decrease i.e. the aggregate interlock will reduce which in turn will decrease the resistance to shear. In another study conducted by Kani in 1966, the influence of concrete strength for the values tested was insignificant on the shear strength. Recent studies by Angelakos in 2001 have shown that concrete strength has negligible effect on the shear strength.

**Reinforcement ratio:**

Researchers have established that the shear strength of a concrete section is influenced by the percentage of longitudinal steel. As the area of steel increases, the length and width of the flexural cracks are reduced. As the crack width is reduced, the amount of aggregate interlock is increased, thereby increasing the shear capacity of the concrete section. Minimizing the length of cracks, results in greater uncracked concrete area to carry shear.

**Depth of beams:**

According to ACI equations, shear strength is directly proportional to the depth of the section. Studies conducted by Kani and Shioya have shown that the shear strength at failure decreases as depth of the section increases. Kani was one of the first researchers to study the size effect. Kani in his paper in 1967 concluded that increasing the beam depth considerably reduces the relative beam strength. He also concluded that shear strength of a beam depends on two other main variables longitudinal reinforcement ratio and shear span to depth ratio.
Later in 1984 Bezant and Kim derived an equation to predict the shear strength of reinforced concrete beams based on fracture mechanics. Their equation accounted for longitudinal reinforcement ratio, aggregate size and depth of beams. This trend is observed in reinforced concrete members since, crack widths are increased with increasing depth. Increased crack widths results in reduction of aggregate interlock thus decreasing the shear carrying capacity of the section.

**Shear span to depth ratio:**

The shear span to depth ratio, a/d, influences the mode of failure. Kani in his paper in 1967 indicated that there exists a clearly defined region bounded by the values of reinforcement ratio and shear span to depth ratio within which diagonal shear failure is imminent, and outside which flexural failures will occur. This phenomenon is independent of the depth of the beams. Kani in his paper in 1967 concluded that shear strength of reinforced concrete beams depends on a/d ratio.

**Applied Axial Loads:**

As axial compression increases, beginning of the flexure crack is delayed thereby increasing the shear strength of the section. Also, axial compression increases, the number of stirrups that can carry the shear across diagonal cracks and axial tension decreases the number of stirrups that can carry shear across the cracks.

**Transverse reinforcement:**

Transverse reinforcement also called as stirrups is used for two reasons (1) to increase the shear strength of concrete beams and (2) to change the mode of failure from explosive
brittle failure to ductile failure. Stirrups are typically perpendicular to longitudinal reinforcement and are spaced at varying intervals depending on shear requirements of the beams. In the past European codes assumed the entire applied shear to be resisted by the transverse reinforcement and neglected the concrete contribution to shear resistance. But the ACI code in America has always included the concrete contribution to shear resistance, which agrees with all the experimental data available from the extensive research conducted.

Before the first diagonal crack occurs, strain in stirrups is equal to the strain in concrete and concrete cracks at a very small strain. Therefore, the stirrups contribute to the shear resistance only after the formation of the diagonal crack. After the formation of the inclined crack the stirrups enhance the beam in the following ways.

1. The stirrups crossing the crack help in resisting the shear force.
2. The stirrups restrict the growth of the cracks and reduce their penetration into the compression zone. This leaves more uncracked concrete available for resistance.
3. The stirrups serve to prevent the cracks from widening. This helps to maintain the aggregate interlock within the concrete.
4. Because the stirrups are tied to the longitudinal reinforcement, they provide extra restraint against the splitting of concrete along the longitudinal bars (Nilson et al., 2004).
2.3 Analysis Methods

The current ACI 318-05 code procedure is based on empirical equations developed from experimental results. The code lacks a physical model for the behavior of beams subject to shear. The current expressions to calculate shear neglect the effect of longitudinal tensile reinforcement and depth of beams. This section presents the different theoretical models.

2.3.1 The Truss Analogy

In 1899, Ritter recommended to use a truss to establish the distribution of forces in a cracked beam. Ritter in his truss model ignored the tensile stresses in cracked concrete. As failure impends due to the action of applied loads, the beam begins to develop inclined cracks. The distribution of forces is shown in Figure 2-2. It can be seen from Figure 2-2 that, the beam develops compressive force in the top flange and tensile forces in the bottom flange. The stirrups are in tension and the concrete between the inclined cracks is in compression.

![Internal forces in a cracked beam](image)

*Figure 2-2: Internal forces in a cracked beam (MacGregor et al., 2005)*
In the imaginary truss shown in Figure 2-3, the upper chord represents the compression zone of the beam, the lower chord represents the longitudinal tension steel. All the vertical stirrups crossing section A-A are lumped together to form one member b-c. All the concrete compressive diagonals crossing section B-B are lumped together to form member e-f (MacGregor et al., 2005). Vertical truss members act as stirrups and the truss diagonals represent the concrete between the inclined cracks. In drawing the truss it is assumed that:

1. The cracks are at an angle 45° with respect to the beam axis.
2. All the shear is resisted by the stirrup and
3. All the stirrups are stressed to their yield point to ensure ductile failure. Therefore the force in any stirrup is given by $A_v f_y$.

![Figure 2-3: Imaginary truss (MacGregor et al., 2005)](image)

As mentioned earlier, Ritter assumed that the applied shear is resisted only by the stirrups and proposed Equation 2-1 to calculate the shear strength of concrete beams.
V = \frac{A_s f_s d \cot \theta}{s} \quad \text{Eqn. 2-1}

Experiments were conducted to verify the Equation 2-1. After experimental analysis it was observed that the measured stress in the stirrups was less than the stress given by the Equation 2-1. It was also observed that the measured and calculated stress varied by a constant amount. So researchers attributed this difference to the concrete contribution to shear strength. Till today the concrete contribution is an empirical equation. Further research has shown that the angle of inclination of the cracks is not 45° but can vary from 25° to 65°. This led to the theory of Variable Angle truss model.

Figure 2-4 shows a variable angle truss model. The variable angle model involves compression fans and compression fields. The compression fans occur at supports or under direct loads, and involves a number of diagonal struts fanning out from the region. The number of struts should be enough such that the vertical load can fully be resisted. The compression field contains diagonal compression struts that run between the compression fields. Like the original truss model, all stirrups are assumed to have yielded (Nilson et al., 2004).
2.3.2 Modified Compression Field Theory

The Modified Compression Field Theory (MCFT) was first presented by Collins and Vecchio (1986). MCFT gives a direct estimation of the crack inclination with respect to the beam axis. MCFT treats cracked concrete as a new material with its own stress-strain properties. In addition, formulations in terms of equilibrium, compatibility, and stress-strain relationships are made with regards to average stresses and average strains. MCFT is an extension of Compression Field Theory (CFT) proposed by Mitchell & Collins in 1974. The difference between MCFT and CFT is that MCFT accounts for tensile strength of concrete after cracking.

If one considers a small concrete element where the longitudinal (x-axis) and transverse (y-axis) coincide with the reinforcement directions, then the element will contain the axial stresses $f_x$ and $f_y$, and the shear stress $v_{xy}$. If the edges remain straight and parallel
upon deformation, then the new shape can be defined by the normal strains $\varepsilon_x$ and $\varepsilon_y$, and the shear strain $\gamma_{xy}$. The Figure 2-5 illustrates the conditions mentioned above.

![Figure 2-5: Stress-strain on a concrete element (Collins & Vecchio, 1986)](image)

Due to compatibility conditions strains in concrete should be equal to strains in steel. The strains in concrete and steel are expressed as average strains even though local conditions may widely vary. Because any deformation in the concrete must be matched by an equal deformation in the steel, a change in concrete strain will show an identical change in steel strain. From Figure 2-6, the following relationships were established where, $\varepsilon_1$ is the principal tensile strain and $\varepsilon_2$ is the principal compressive strain.

$$\lambda_\gamma = \frac{2(\varepsilon_1 - \varepsilon_2)}{\tan \theta}$$

Eq. 2-2
The forces applied to the concrete element are resisted by stresses in both the concrete and the steel reinforcement. The following equilibrium equations were derived:

\[ f_x = f_{cx} + \rho_{sx} \cdot f_{sx} \]  
Eq. 2-5

\[ f_y = f_{cy} + \rho_{sy} \cdot f_{sy} \]  
Eq. 2-6

\[ v_{xy} = v_{cx} + \rho_{sx} \cdot v_{sx} \]  
Eq. 2-7

\[ v_{xy} = v_{cy} + \rho_{sy} \cdot v_{sy} \]  
Eq. 2-8
Assuming, \( v_{xy} = v_{cy} = v_{cxy} \) then the concrete stress conditions are fully defined if \( f_{cx}, f_{cy}, \) and \( v_{cxy} \) are known. The concrete element will resist concrete shear forces \( v_{cx} \), horizontal concrete stresses \( f_{cx} \), and vertical concrete stresses \( f_{cy} \). The average concrete stresses are shown in Figure 2-7a. These average stresses on concrete are combined to form the principal tensile stress \( f_{c1} \) and the principal compressive stress \( f_{c2} \). This is shown in Figure 2-7b. Using the Mohr’s Circle shown in Figure 2-7c, we can further define:

\[
f_{cx} = f_{c1} - \frac{v_{cxy}}{\tan \theta_c} \quad \text{Eq. 2-9}
\]

\[
f_{cy} = f_{c1} - v_{cxy} \tan \theta_c \quad \text{Eq. 2-10}
\]

\[
f_{c2} = f_{c1} - v_{cxy} \left( \tan \theta_c + \frac{1}{\tan \theta_c} \right) \quad \text{Eq. 2-11}
\]
Figure 2-7: Average stresses in a cracked element (Collins & Vecchio, 1986)

A major assumption in the MCFT is that the principal stress axes and the principal strain axes coincide by the angle $\theta$. Also, the stress-strain relationship for the reinforcement is assumed to be bilinear. Collins and Vecchio (1986), developed the following average stress - average strain response of concrete. It is interesting to note that the principal compressive stress $f_{c2}$ is both a function of the principal compressive strain $\varepsilon_2$ and of the principal tensile strain $\varepsilon_1$. 

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19
In Eq. 2-13 $\varepsilon_c^\prime$ is the strain in a concrete cylinder at peak stress and is equal to -0.002. This causes $f_{c,2\text{max}}$ to decrease with increasing tensile strain. The relationship for the average principal tensile strain is linear up until cracking, as expressed in the formulas below.

$$f_{c1} = E_c \varepsilon_1 \quad \text{if} \quad \varepsilon_1 \leq \varepsilon_{cr}$$  \hspace{1cm} \text{Eq. 2-14}

$$f_{c1} = \frac{f_{cr}}{1 + \sqrt{200 \varepsilon_1}} \quad \text{if} \quad \varepsilon_1 > \varepsilon_{cr}$$  \hspace{1cm} \text{Eq. 2-15}

After cracking occurs in a beam shear is carried by the aggregate interlock mechanism. The diagonal crack width is computed by multiplying the principal tensile strain by the crack spacing $s_\theta$, as shown below.

$$w = \varepsilon_1 s_\theta$$  \hspace{1cm} \text{Eq. 2-16}

$$s_\theta = \frac{1}{\sin \theta + \cos \theta} \left( \frac{s_{mx}}{s_{ny}} \right)$$  \hspace{1cm} \text{Eq. 2-17}

Collins and Vecchio suggested various solution techniques for the MCFT. All the methods are tedious to calculate by hand. The rigorous procedure requires individual concrete layers and reinforcing bar elements to be analyzed separately along the entire cross section. In 1996, Collins et al. created a simple procedure for the design of shear which
utilizes the assumption that shear stress is to remain constant over the depth of the web. This procedure later formed the basis for the AASHTO LRFD Bridge Design Specifications.

2.3.3 Simplified Modified Compression Field Theory

In 2006, Bentz et al. presented a more straightforward approach to the MCFT and entitled it the simplified modified compression field theory (SMCFT). Their goal was to provide very accurate shear calculations while giving a less complicated method than the MCFT. The authors also drew on prior research, such as work done by Collins et al. (1996), to complete many of the design equations. While describing the method, please note that the authors changed their notation for the y-axis to be called the z-axis. Figure 2-8 gives a summary of the equations derived from the MCFT.
The clamping stresses $f_z$ are assumed to be negligibly small since the element is modeled in the flexural region. If failure occurs before the transverse reinforcement yields, then the shear stress level will be $0.25 f_y'$. For failures below this level, it will be assumed that $f_{sz}$ and $f_{scrr}$ are equal to the yield stress of the transverse reinforcement $f_y$. From the equations originally presented by the modified compression field theory, the following results can be made.

$$v = v_{ci} + \rho_z f_y \cot \theta$$  \hspace{1cm} \text{Eq. 2-18}
\[ v = f_i \cot \theta + \rho f_y \cot \theta \]  
\text{Eq. 2-19}

\[ v = v_c + v_i = \beta \sqrt{f_c} + \rho f_y \cot \theta \]  
\text{Eq. 2-20}

The value \( \beta \) was originally defined by Collins et al. (1996) as the tensile stress factor. Based on calculations for \( \beta \) and \( \theta \) without transverse reinforcement, as well as fitting comparisons with the MCFT, the following final relations were made.

\[ \beta = \frac{0.4}{1+1500\varepsilon_x} \times \frac{51}{39+s_{sc}} \]  
\text{Eq. 2-21}

\[ \theta = \left( 29 \deg + 7000\varepsilon_x \right) \left( 0.88 + \frac{s_{sc}}{100} \right) \leq 75 \deg \]  
\text{Eq. 2-22}

The stress across the cracks can be computed by:

\[ f_{scr} = \frac{(v + v_c)\cot \theta}{\rho_x} \]  
\text{Eq. 2-23}

### 2.4 Code Provisions

This section presents common design codes currently used to predict the shear capacity of reinforced concrete beams. Three different codes are presented in this study to estimate the shear strength of beams. ACI is based on empirical equations to estimate the shear strength and they equations are still based on the 45° truss model while others like AASHTO and CSA rely on more exact solutions like Modified Compression Field Theory and Simplified Modified Compression Field Theory.
2.4.1 American Concrete Institute, ACI 318-05

At present, ACI codes use empirical expressions developed from various experiments to predict the shear strength of a reinforced concrete members. ACI has always assumed that a part of the shear strength has been resisted by concrete and the rest is carried by transverse reinforcement. The steel contribution to shear strength is based on the 45° truss model. These equations neglect the size effect and effect of longitudinal reinforcement ratio in shear strength of reinforced concrete beams. The shear strength is based on an average shear stress distribution across the entire cross section.

The basic design equation for the shear strength of concrete beams is

\[
\Phi V_n \geq V_u \\
\text{Eq. 2-24}
\]

where,

\(\Phi\) is the strength reduction factor, which is equal to 0.75 for shear,

\(V_u\) is the factored shear force at the section considered and

\(V_n\) is the nominal shear strength computed by,

\[
V_n = V_c + V_s \\
\text{Eq. 2-25}
\]

where,

\(V_c\) is the concrete contribution to the shear strength and

\(V_s\) is the nominal shear strength provided by shear reinforcement, based on the 45° truss.

In 1962, the ACI – ASCE Committee 326 proposed the following equation to calculate the concrete contribution to shear strength, \(V_c\).
\[ V_c = \left( 1.9\sqrt{f'_c} + 2500\rho_w \frac{V_u}{M_u} \right) b_w d \leq 3.5\sqrt{f'_c} b_w d \]

Eq. 2-26

As the above equation is quite tedious to apply, ACI 318-05 allows the following simpler equation to be used.

\[ V_c = 2\sqrt{f'_c} b_w d \]

Eq. 2-27

The shear resisted by the stirrups can be calculated by the equation:

\[ V_s = \frac{A_v f_y d}{s} \]

Eq. 2-28

\[ A_{v,\text{min}} = 0.75\sqrt{f'_c} \frac{b_w s}{f_{y,t}} \geq \frac{50b_w s}{f_{y,t}} \]

Eq. 2-29

where,

- \( f'_c \) is the concrete cylinder compressive strength in pounds per square inch
- \( \rho_w \) is the ratio of \( A_v \) to \( b_w d \)
- \( A_v \) is the area of non-prestressed longitudinal tension reinforcement in square inch
- \( b_w \) is the web width in inches
- \( d \) is the distance from extreme compression fiber to centroid of longitudinal tension reinforcement in inches
- \( V_u \) is the factored shear at section in pounds
- \( M_u \) is the factored moment at section in pound – inch
- \( A_v \) is the area of shear reinforcement in square inch
- \( f_{y,t} \) is the specified yield strength \( f_y \) of transverse reinforcement in pounds per square inch
s is the center-to-center spacing of transverse reinforcement in inches

2.4.2 CSA A23.3 – 04

The latest Canadian Standards for shear strength is based on the Modified Compression Field Theory. The equations for the evaluation of factored shear resistance are given below.

To ensure adequate shear resistance is provided,

\[ V_r \geq V_f \]  \hspace{1cm} \text{Eq. 2-30}

where,

\( V_r \) is the factored shear resistance and

\( V_f \) is the factored shear force

For a non-prestressed section, the factored shear resistance is given by:

\[ V_r = V_c + V_s \]  \hspace{1cm} \text{Eq. 2-31}

To prevent the crushing of concrete in web prior to yielding of web reinforcement, \( V_r \) shall not exceed,

\[ V_{r,max} = 0.25 \phi \sqrt{f'_c b_n d_v} \]  \hspace{1cm} \text{Eq. 2-32}

The value of \( V_c \) is calculated from,

\[ V_c = \phi \lambda f'_c b_n d_v \]  \hspace{1cm} \text{Eq. 2-33}

\[ \beta = \frac{0.4}{1 + 1500 \varepsilon_x} \cdot \frac{51}{39 + S_{se}} \]  \hspace{1cm} \text{Eq. 2-34}
For members with transverse reinforcement perpendicular to the longitudinal axis, $V_s$ is calculated from,

$$V_s = \frac{\phi_s A, f_y d, \cot \theta}{s}$$  \hspace{1cm} \text{Eq. 2-36}

$$\theta = 29 + 7000 \varepsilon_x$$  \hspace{1cm} \text{Eq. 2-37}

$$A_{v, \text{min}} = 0.72 \sqrt{f'_c} \frac{b_s}{f_y}$$  \hspace{1cm} \text{Eq. 2-38}

where,

$\Phi_c$ is the material factor for concrete equal to 0.6

d, is the effective shear depth, taken as greater of 0.9d or 0.72h

$\lambda$ is the factor to account for low density concrete ($\lambda = 1$ for normal density concrete)

$\beta$ is the factor accounting for shear resistance of cracked concrete

$\Phi_s$ is the material factor for steel equal to 0.85

$\theta$ is the angle of inclination of diagonal compressive stresses to the longitudinal axis of the member

$\varepsilon_x$ is the longitudinal strain at mid-depth of the member due to factored loads

$f'_c$ is the concrete cylinder compressive strength in kilo pound per square inch
2.4.3 AASHTO LRFD – 04

The AASHTO LRFD proposes a more rational approach based on fundamental principles (Modified Compression Field Theory) rather than empirical equations.

For non-prestressed sections, the nominal shear resistance $V_n$ can be determined as the lesser of:

$$V_n = V_c + V_s \quad \text{Eq. 2-39}$$

$$V_{n,max} = 0.25 f'_c b_v d_v \quad \text{Eq. 2-40}$$

$$V_c = 0.0316 \beta \sqrt{f'_c b_v d_v} \quad \text{Eq. 2-41}$$

$$V_s = \frac{A_v f_v d_v \cot \theta}{s} \quad \text{Eq. 2-42}$$

The minimum area of transverse reinforcement is given by,

$$A_{v,min} \geq 0.0316 \sqrt{f'_c \frac{b_v s}{f_y}} \quad \text{Eq. 2-43}$$

where,

$b_v$ is the effective web width, taken as the minimum web width within the depth

d_v is the effective shear depth = the greater of 0.9d or 0.72h

$\beta$ is the factor indicating the ability of diagonally cracked concrete to transmit tension

$\theta$ is the angle of inclination of diagonal compressive struts

$f'_c$ is the concrete compressive strength kilo pound per square inch

$f_y$ is the yield strength of the transverse reinforcement kilo pound per square inch
**Determination of β and θ:**

For section containing at least the minimum amount of transverse reinforcement, the values of β and θ can be determined from Table 2-1. For the use of Table 2-1, the designer should have knowledge of the shear design stress ratio \( \frac{\nu}{f'_c} = \frac{V_u}{b_i d_i f_c} \) and the longitudinal strain, \( \varepsilon_x \), at mid-depth. \( \varepsilon_x \) can be determined using the following expression for Table 1:

\[
\varepsilon_x = \frac{\frac{M}{d_i} + 0.5N_u + 0.5(V_u - V_p) \cot \theta - A_p f_p}{2(E_i A_i + E_p A_p)}
\]

Eq. 2-44

After calculating the two values, shear design stress ratio and longitudinal strain \( \varepsilon_x \), the designer should select the row corresponding to shear design stress ratio and select the column corresponding to longitudinal strain in Table 2-1.

For section containing less transverse reinforcement than specified by \( A_{v,\text{min}} \), the values of β and θ can be determined from Table 2-2. To use Table 2-2 the designer should have knowledge of equivalent spacing parameter \( S_{xe} \) and the longitudinal strain, \( \varepsilon_x \), at mid-depth. \( S_{xe} \) and \( \varepsilon_x \) for Table 2-2 can be determined using the following expressions:

\[
s_{xe} = \frac{1.38 s_x}{a_x + 0.63}
\]

Eq. 2-45

\[
\varepsilon_x = \frac{\frac{M}{d_i} + 0.5N_u + 0.5(V_u - V_p) \cot \theta - A_p f_p}{E_i A_i + E_p A_p}
\]

Eq. 2-46

If the value of \( \varepsilon_x \) from the above two equations is negative then,
\[
\varepsilon_x = \frac{M_u}{d_i} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{ps} - \frac{2(E_cA_c + E_sA_s + E_pA_p)}{E_cA_c + E_sA_s + E_pA_p}
\]

Eq. 2-47

where,

\(A_c\) is the area of concrete on the flexure tension side of the member in square inch

\(A_{ps}\) is the area of pre-stressing steel on the flexure tension side of the member in square inch

\(f_{ps}\) is a parameter taken as modulus of elasticity of pre-stressing tendons multiplied by the locked in strain between the pre-stressing tendons and the surrounding concrete in kilo pound per square inch

\(N_u\) is the factored axial force, taken as positive if tensile and negative if compressive in kilo pound

\(V_p\) is the component in the direction of the applied shear of the effective pre-stressing force in kilo pound.
Table 2-1: Members with at least minimum shear reinforcement (NCHRP, 2005)

<table>
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<tr>
<th>( \frac{V^*}{f'c} )</th>
<th>Longitudinal Strain, ( \varepsilon_y \times 1000 )</th>
<th>( \leq -0.20 )</th>
<th>( \leq -0.10 )</th>
<th>( \leq -0.05 )</th>
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<th>( \leq 0.50 )</th>
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<td>1.58</td>
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</table>

\( ^* V = V / b_h d_v \)
Table 2-2: Members with less than minimum shear reinforcement (NCHRP, 2005)

| $s_{av}$ \ (in) | \(\leq -0.20\) | \(\leq -0.10\) | \(\leq -0.05\) | \(\leq 0\) | \(\leq 0.125\) | \(\leq 0.25\) | \(\leq 0.50\) | \(\leq 0.75\) | \(\leq 1.00\) | \(\leq 1.50\) | \(\leq 2.00\) |
|-----------------|-----------------|-----------------|-----------------|---------|-----------------|---------|-----------------|---------|-----------------|---------|-----------------|---------|
| \(\theta\)     | 25.4°           | 25.5°           | 25.9°           | 26.4°   | 27.7°           | 28.9°   | 30.9°           | 32.4°   | 33.7°           | 35.6°   | 37.2°           | 37.2°   |
| \(\beta\)      | 6.36            | 6.06            | 5.56            | 5.15    | 4.41            | 3.90    | 3.26            | 2.86    | 2.58            | 2.21    | 1.96            |
| \(\leq 10\)    | 27.6°           | 27.6°           | 28.3°           | 29.3°   | 31.6°           | 33.5°   | 36.3°           | 38.4°   | 40.1°           | 42.7°   | 44.7°           | 44.7°   |
| \(\beta\)      | 5.78            | 5.78            | 5.38            | 4.89    | 4.05            | 3.52    | 2.88            | 2.50    | 2.23            | 1.88    | 1.65            |
| \(\leq 15\)    | 29.5°           | 29.5°           | 29.7°           | 31.1°   | 34.1°           | 36.5°   | 39.9°           | 42.4°   | 44.4°           | 47.4°   | 49.7°           | 49.7°   |
| \(\beta\)      | 5.34            | 5.34            | 5.27            | 4.73    | 3.82            | 3.27    | 2.64            | 2.27    | 2.01            | 1.68    | 1.46            |
| \(\leq 20\)    | 31.2°           | 31.2°           | 31.2°           | 32.3°   | 36.0°           | 38.8°   | 42.7°           | 45.5°   | 47.6°           | 50.9°   | 53.4°           | 53.4°   |
| \(\beta\)      | 4.99            | 4.99            | 4.99            | 4.61    | 3.65            | 3.09    | 2.46            | 2.09    | 1.85            | 1.52    | 1.31            |
| \(\leq 30\)    | 34.1°           | 34.1°           | 34.1°           | 34.2°   | 38.9°           | 42.3°   | 46.9°           | 50.1°   | 52.6°           | 56.2°   | 59.0°           | 59.0°   |
| \(\beta\)      | 4.46            | 4.46            | 4.46            | 4.43    | 3.39            | 2.82    | 2.19            | 1.84    | 1.61            | 1.30    | 1.10            |
| \(\leq 40\)    | 36.6°           | 36.6°           | 36.6°           | 36.6°   | 41.1°           | 45.0°   | 50.2°           | 53.7°   | 56.3°           | 60.2°   | 63.0°           | 63.0°   |
| \(\beta\)      | 4.06            | 4.06            | 4.06            | 4.06    | 3.20            | 2.62    | 2.00            | 1.66    | 1.43            | 1.14    | 0.95            |
| \(\leq 60\)    | 40.8°           | 40.8°           | 40.8°           | 40.8°   | 44.5°           | 49.2°   | 55.1°           | 58.9°   | 61.8°           | 65.8°   | 68.6°           | 68.6°   |
| \(\beta\)      | 3.50            | 3.50            | 3.50            | 3.50    | 2.92            | 2.32    | 1.72            | 1.40    | 1.18            | 0.92    | 0.75            |
| \(\leq 80\)    | 44.3°           | 44.3°           | 44.3°           | 44.3°   | 47.1°           | 52.3°   | 58.7°           | 62.8°   | 65.7°           | 69.7°   | 72.4°           | 72.4°   |
| \(\beta\)      | 3.10            | 3.10            | 3.10            | 3.10    | 2.71            | 2.11    | 1.52            | 1.21    | 1.01            | 0.76    | 0.62            |

\[ s_{av} = \frac{1.38s_s}{0.63 + a_g} \ (in. \ units), \] where \(s_s\) is the lesser of \(d_a\) and the maximum distance between layers of crack control reinforcement (in.), \(a_g\) = maximum aggregate size (in.).
2.5 High Performance steel – MMFX

As mentioned in Chapter 1 the high strength high performance steel used in this research is MMFX (Micro-composite Multi-structural Formable Steel) which has been developed and produced by MMFX Steel Corporation of America through the use of nanotechnology. The company controls the material properties at the atomic level with the aid of the electron microscope. MMFX steel has a unique microstructure formed during the production process, which results in superior corrosion resistance and mechanical properties when compared to conventional Grade 60 steel in addition to its equal and in many cases superior properties of strength, toughness, energy absorption and formability. This distinctive physical feature minimizes the formation of galvanic cells in steel, which is a substantial contributor to the initiation and acceleration of corrosion. As a result, corrosion activity can either be eliminated or minimized (MMFX, 2002).

This highly corrosion resistant steel increases the serviceability and life of structures. Also, high design yield strength allows for reduction of reinforcement area without reducing ultimate load carrying capacities of a structure and permanently deforming the structure. The reduced reinforcement ratio in a concrete member also eliminates the problem of rebar congestion.

The mechanical properties of MMFX steel relevant to structural engineering are well documented. MMFX steels are inherently stronger than conventional steel, while maintaining a good level of ductility (El-Hacha, 2002). The engineering stress-strain curve of MMFX is linear with a modulus of elasticity of 29,000 ksi up to 95 ksi. The curve becomes nonlinear at
a proportional limit of approximately 95 ksi and does not have a well-defined yield plateau. As MMFX does not have a well defined yield plateau, the minimum specified yield strength is estimated using the 0.2% offset method. The yield strength determined using this method is approximately 100 ksi while conventional steels have 60 ksi. Figure 2-8 shows the stress strain curve of the MMFX steel in comparison to A615 Grade 60 steel.

![Stress-strain curve](image)

**Figure 2-8: Typical stress-strain curve**

Ultimate tensile stress of MMFX bars has been reported in the range of 165 to 175 ksi which is more than double the ultimate stress typically achieved from testing conventional grade 60 reinforcing bars.
2.5.1 MMFX as Shear Reinforcement

This section presents the research conducted by Sumpter (2007) at North Carolina State University, Raleigh. The primary objective of this research was to determine the feasibility of using high performance steel as shear reinforcement for concrete beams.

Nine reinforced concrete beams were used to determine the shear behavior. Shear span to depth ratio, stirrup spacing and type of steel material were the different parameters studied in this research. The MMFX transverse reinforcement in this research was not designed for yield strength of 80 ksi. The conventional Grade 60 steel was only substituted by MMFX rebars. Among the nine beams, three beams were fabricated with conventional Grade 60 longitudinal reinforcement and MMFX stirrups. From the experimental data, analysis of the data and analytical modeling it was concluded that:

1. For the three beams reinforced with Grade 60 longitudinal reinforcement and MMFX transverse reinforcement, there was only a slight increase in the shear strength and serviceability of these beams.

2. Direct replacement of both longitudinal and transverse conventional Grade 60 steel with MMFX steel increased the capacity of flexural members and enhanced the serviceability by reducing the crack width.

3. The high strength characteristics of MMFX steel could change the failure mode from a shear failure to flexural failure.
3. EXPERIMENTAL PROGRAM

This chapter describes the experimental program completed at the Constructed Facilities Laboratory, North Carolina State University in Raleigh to study the shear behavior of concrete members reinforced with MMFX rebars.

The primary objective of this experimental program is to utilize the strength of Micro-composite Multi-structural Formable Steel (MMFX) as shear reinforcement for reinforced concrete flexure members at selected yield strengths of 80 ksi and 100 ksi. The main variables used in the current study are the longitudinal and transverse reinforcement ratio and the type of steel used for longitudinal and transverse reinforcement. All beams reinforced with shear reinforcement were designed to achieve the same ultimate nominal shear capacity. This criteria lead to different shear reinforcement ratio for each beam, depending on the selected yield strength of the MMFX stirrups.

3.1 Design of Test Specimens

The experimental program comprised a total of eighteen tests on nine large-sized beams, tested under static loading condition up to failure. All specimens were large-sized dimensions to simulate field behavior of typical concrete beams. The nine beams were classified into three groups based on their shear resistance. Within each group, the beams were geometrically similar. The cross section configurations were designed in consideration of the equipment and limitations at the laboratory. All beams have total length of 22 ft., and
were designed using nominal concrete compressive strength of 4000 psi. The total length was chosen in order to have the ability to test each beam twice, and thus double the amount of collected data. The shear span-to-depth ratio, a/d, of all specimens was kept constant at a/d=3.0. The spacing of shear reinforcement was varied to reflect a minimum and maximum level of shear resistance. Figures 3-1 to 3-3 shows the sectional elevation and cross section for Groups 1 to 3 beams respectively. A summary of the groups and the cross-sections of the specimens are given in Table 3-1.

Figure 3-1: Typical section of Group 1 beams
Figure 3-2: Typical section of Group 2 beams

Figure 3-3: Typical section of Group 3 beams
Test specimens are designed to induce stresses of 80 ksi and 100 ksi in the MMFX stirrups. Within each group, beams reinforced with stirrups are designed to achieve the same ultimate nominal shear capacity. In each group the beams reinforced with MMFX stirrups are compared with beams reinforced with conventional Grade 60 steel stirrups. Beams G1-M0, G2-M0, G3-C0 and G3-M0 are designed without shear reinforcement to accurately estimate the nominal shear strength provided by concrete, $V_c$. 
Table 3-1: Reinforcement details of beams

<table>
<thead>
<tr>
<th>Group</th>
<th>ID</th>
<th>Target Shear Capacity</th>
<th>Cross Section</th>
<th>a/d</th>
<th>Design Flexural Stress</th>
<th>Flexural Steel</th>
<th>Design Stirrup Stress</th>
<th>Stirrup Size</th>
<th>Spacing</th>
<th>in</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>G1-M0</td>
<td>Min. 3(\sqrt[3]{f_{c, bd}})</td>
<td>24 x 23</td>
<td>3.1</td>
<td>MIMFX 100 ksi</td>
<td>4 # 11 2 # 9</td>
<td>-</td>
<td>-</td>
<td>8.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>G1-C0</td>
<td>Min. 3(\sqrt[3]{f_{c, bd}})</td>
<td>24 x 23</td>
<td>3.1</td>
<td>MIMFX 100 ksi</td>
<td>6 # 11 2 # 9</td>
<td>Conv. 60 ksi</td>
<td>#3</td>
<td>8.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>G1-M80</td>
<td>Min. 3(\sqrt[3]{f_{c, bd}})</td>
<td>24 x 23</td>
<td>3.1</td>
<td>MIMFX 100 ksi</td>
<td>4 # 11 2 # 9</td>
<td>MIMFX 80 ksi</td>
<td>#3</td>
<td>10.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>G1-M100</td>
<td>Min. 3(\sqrt[3]{f_{c, bd}})</td>
<td>24 x 23</td>
<td>3.1</td>
<td>MIMFX 100 ksi</td>
<td>4 # 11 2 # 9</td>
<td>MIMFX 100 ksi</td>
<td>#3</td>
<td>13.0</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>G2-M0</td>
<td>Min. 3(\sqrt[3]{f_{c, bd}})</td>
<td>24 x 23</td>
<td>3.1</td>
<td>MIMFX 100 ksi</td>
<td>4 # 11 2 # 9</td>
<td>-</td>
<td>-</td>
<td>8.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>G2-C0</td>
<td>Min. 3(\sqrt[3]{f_{c, bd}})</td>
<td>24 x 23</td>
<td>3.1</td>
<td>MIMFX 100 ksi</td>
<td>6 # 11 2 # 9</td>
<td>Conv. 60 ksi</td>
<td>#3</td>
<td>8.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>G2-M80</td>
<td>Min. 3(\sqrt[3]{f_{c, bd}})</td>
<td>24 x 23</td>
<td>3.1</td>
<td>MIMFX 100 ksi</td>
<td>4 # 11 2 # 9</td>
<td>MIMFX 80 ksi</td>
<td>#3</td>
<td>10.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>G2-M100</td>
<td>Min. 3(\sqrt[3]{f_{c, bd}})</td>
<td>24 x 23</td>
<td>3.1</td>
<td>MIMFX 100 ksi</td>
<td>4 # 11 2 # 9</td>
<td>MIMFX 100 ksi</td>
<td>#3</td>
<td>13.0</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>G3-C0</td>
<td>Max. 7(\sqrt[3]{f_{c, bd}})</td>
<td>16 x 22</td>
<td>3.0</td>
<td>Conv. 60 ksi</td>
<td>7 # 11 4 # 10</td>
<td>-</td>
<td>-</td>
<td>4.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>G3-M0</td>
<td>Max. 7(\sqrt[3]{f_{c, bd}})</td>
<td>16 x 22</td>
<td>3.0</td>
<td>MIMFX 100 ksi</td>
<td>5 # 11 4 # 10</td>
<td>-</td>
<td>-</td>
<td>4.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>G3-C0</td>
<td>Max. 7(\sqrt[3]{f_{c, bd}})</td>
<td>16 x 22</td>
<td>3.0</td>
<td>Conv. 60 ksi</td>
<td>7 # 11 4 # 10</td>
<td>Conv. 60 ksi</td>
<td>#4</td>
<td>4.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>G3-M80</td>
<td>Max. 7(\sqrt[3]{f_{c, bd}})</td>
<td>16 x 22</td>
<td>3.0</td>
<td>MIMFX 100 ksi</td>
<td>5 # 11 4 # 10</td>
<td>MIMFX 80 ksi</td>
<td>#4</td>
<td>5.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>G3-M100</td>
<td>Max. 7(\sqrt[3]{f_{c, bd}})</td>
<td>16 x 22</td>
<td>3.0</td>
<td>MIMFX 100 ksi</td>
<td>5 # 11 4 # 10</td>
<td>MIMFX 100 ksi</td>
<td>#4</td>
<td>7.0</td>
<td></td>
</tr>
</tbody>
</table>

The beams, shown in Table 3-1 are identified according to three parameters: the first two characters indicate the group to which the beam belongs, i.e. G1 is Group 1, G2 is Group 2 and G3 is Group 3. The second parameter specifies the longitudinal and transverse steel type i.e. C is for conventional steel and M is for MMFX steel. The third parameter is the design stress in the stirrup, 0 indicates no transverse reinforcement, 60 indicates 60 ksi, 80 is
80 ksi and 100 is 100 ksi design stress in the stirrup. For example, G2-M80 is the designation for a beam in group 2 with MMFX longitudinal and transverse reinforcement and the stirrups are designed to achieve 80 ksi stress. A further identification will be used in this thesis to distinguish between first test and second test. The additional identification will be ‘R’ for the repeated test. This identification will be added to the end of the specimen name. This identification will be used only in Group 3. For the remaining four beams with target shear capacity of $3√f'_c\,bd$, the first test is categorized as Group 1 and the second test is categorized as Group 2. The difference in these two groups is the test setup configuration. The detailed description of the test setup configuration is provided in Section 3.5.

It can be seen from Table 3-1 that, beams are classified into three main categories depending on their shear resistance. Group 1 and Group 2 are identical except the test setup configuration. Table 3-1 also presents the following details, the design stress in the longitudinal bars, no. of bars at the top (compression) and bottom (tension) of the beam, design stress in the stirrups, stirrup size and the spacing of stirrups in each beam.

The first and the second group represents beams that are designed to have a nominal shear resistance, $V_n$, equivalent to $3√f'_c\,bd$. The shear reinforcement is controlled by the maximum allowable spacing, $d/2$, specified by the ACI 318-05 and the minimum allowable transverse reinforcement bar size, #3bars, for both the MMFX and the Grade 60 Steel.

Group 3 beams, were developed to have shear resistance within the range of $7√f'_c\,bd$. The maximum shear reinforcement level is based on the allowable spacing, $d/4$, given in the ACI 318-05.
The beams were designed to avoid premature flexural failure, therefore, achieves the design shear strength. Shear strengths were calculated according to ACI 318-05 Eq. 11-3 and 11-15. Once the shear capacity of the beam was determined, flexural analysis was performed using cracked section analysis. Adequate longitudinal reinforcements were provided to avoid the pre-mature flexural failure and to allow the high performance stirrups to achieve a stress level of 80 ksi and 100 ksi. Reinforcement details of the beams for the experimental program are provided in Table 3-1. Table 3-2 shows the percentage reduction of longitudinal and transverse steel in beams reinforced with MMFX steel when compared to beams reinforced with conventional steel. Table 3-3 summarizes the design characteristics of each beam.

**Table 3-2: Steel percentage Reduction**

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Transverse Steel</th>
<th>Longitudinal Steel</th>
<th>% Reduced</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Spacing (in)</td>
<td>No. of #11 bars</td>
<td>Transverse Steel</td>
</tr>
<tr>
<td>1</td>
<td>G1-C60</td>
<td>8.0</td>
<td>6</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>G1-M80</td>
<td>10.0</td>
<td>4</td>
<td>25%</td>
</tr>
<tr>
<td></td>
<td>G1-M100</td>
<td>13.0</td>
<td>4</td>
<td>30%</td>
</tr>
<tr>
<td>2</td>
<td>G2-C60</td>
<td>8.0</td>
<td>6</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>G2-M80</td>
<td>10.0</td>
<td>4</td>
<td>25%</td>
</tr>
<tr>
<td></td>
<td>G2-M100</td>
<td>13.0</td>
<td>4</td>
<td>30%</td>
</tr>
<tr>
<td>3</td>
<td>G3-C60</td>
<td>4.5</td>
<td>7</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>G3-C60-R</td>
<td>4.5</td>
<td>7</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>G3-M80</td>
<td>5.5</td>
<td>5</td>
<td>22%</td>
</tr>
<tr>
<td></td>
<td>G3-M80-R</td>
<td>5.5</td>
<td>5</td>
<td>22%</td>
</tr>
<tr>
<td></td>
<td>G3-M100</td>
<td>7.0</td>
<td>5</td>
<td>56%</td>
</tr>
<tr>
<td></td>
<td>G3-M100-R</td>
<td>7.0</td>
<td>5</td>
<td>56%</td>
</tr>
</tbody>
</table>
Table 3-3: Design of beams

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Cross - Section</th>
<th>Spacing, S</th>
<th>Shear Capacity from ACI, V_n</th>
<th>Cracked Section Analysis</th>
<th>( V_n / (?f'_c b d) )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>in</td>
<td>in</td>
<td>kip</td>
<td>kip-in</td>
<td>kip</td>
</tr>
<tr>
<td>1</td>
<td>G1-M0</td>
<td>24 x 28</td>
<td>-</td>
<td>77</td>
<td>13974</td>
<td>177</td>
</tr>
<tr>
<td></td>
<td>G1-C60</td>
<td></td>
<td>8.0</td>
<td>119</td>
<td>12733</td>
<td>161</td>
</tr>
<tr>
<td></td>
<td>G1-M80</td>
<td></td>
<td>10.0</td>
<td>122</td>
<td>13974</td>
<td>177</td>
</tr>
<tr>
<td></td>
<td>G1-M100</td>
<td></td>
<td>13.0</td>
<td>120</td>
<td>13974</td>
<td>177</td>
</tr>
<tr>
<td>2</td>
<td>G2-M0</td>
<td>24 x 28</td>
<td>-</td>
<td>77</td>
<td>13974</td>
<td>177</td>
</tr>
<tr>
<td></td>
<td>G2-C60</td>
<td></td>
<td>8.0</td>
<td>119</td>
<td>12733</td>
<td>161</td>
</tr>
<tr>
<td></td>
<td>G2-M80</td>
<td></td>
<td>10.0</td>
<td>122</td>
<td>13974</td>
<td>177</td>
</tr>
<tr>
<td></td>
<td>G2-M100</td>
<td></td>
<td>13.0</td>
<td>120</td>
<td>13974</td>
<td>177</td>
</tr>
<tr>
<td>3</td>
<td>G3-C0</td>
<td>16 x 22</td>
<td>-</td>
<td>36</td>
<td>9833</td>
<td>182</td>
</tr>
<tr>
<td></td>
<td>G3-M0</td>
<td></td>
<td>-</td>
<td>36</td>
<td>10569</td>
<td>196</td>
</tr>
<tr>
<td></td>
<td>G3-C60</td>
<td></td>
<td>4.5</td>
<td>132</td>
<td>9833</td>
<td>182</td>
</tr>
<tr>
<td></td>
<td>G3-M80</td>
<td></td>
<td>5.5</td>
<td>141</td>
<td>10569</td>
<td>196</td>
</tr>
<tr>
<td></td>
<td>G3-M100</td>
<td></td>
<td>7</td>
<td>139</td>
<td>10569</td>
<td>196</td>
</tr>
</tbody>
</table>
3.2 Material properties

In this section, concrete and steel mechanical properties are reported. The purpose of the material testing program was to determine the material properties so that they can be used to help interpret the behavior of the beams.

3.2.1 Concrete

The concrete was delivered by the local ready-mix concrete supplier. The concrete used Type 1 Portland cement with a maximum aggregate size of 3/8”. This aggregate size was chosen to ensure proper flow of concrete around the rebar. Concrete cylinders were prepared each time beam was cast. Concrete cylinders were stored beside the specimens under the same environmental conditions to obtain consistent results. Five concrete mixtures were used for nine specimens. During each cast, fifteen 4x8 in. cylinders were cast and tested following ASTM procedures to estimate the concrete compressive strength at 7, 14 and 28 days and on the day of the testing. Three 4x8 in. concrete cylinders were used to obtain the concrete compressive strength during the testing of specimens.

Cylinders were tested using a 500 kip capacity Forney compression testing machine in accordance with ASTM C39 at a rate of 35 to 50 psi per second. Neoprene caps were placed on the ends of each cylinder to ensure an even distribution of pressure between the loading surfaces of the testing machine and the cylinder surfaces. The maximum load achieved was used to calculate the ultimate stress of each cylinder. The Forney compression testing machine is shown in Figure 3-4 for cylinder testing.
3.2.2 Steel

Tension tests were performed on reinforcing steel coupons to determine the stress-strain characteristics. Samples of #3 and #4 MMFX and Grade 60 bars were taken from the supply used to construct the beams. The #11 bars were not tested due to limited capacity of the MTS machine.

For each steel type and bar size, four coupons were tested using a MTS universal tension-compression testing machine. Each specimen was subjected to a gradually increasing uniaxial load until failure. Figure 3-5 shows a bar during tension test in a MTS machine.
3.3 Specimen fabrication

This section describes in brief the details of the specimen construction process. The test specimens were constructed at the Constructed Facilities Laboratory. Depending on the beam sizes and reinforcement provided for each beam, bar bending schedules were prepared separately for Conventional Grade 60 steel and MMFX steel. Grade 60 steel was supplied by a local supplier and MMFX steel was supplied by MMFX Steel Corporation based on the bar bending schedule. The bar bending schedule for all the beams is provided in Appendix A. First, the reinforcement cages were assembled outside the forms. During the fabrication of
the cages, the stirrup spacing was carefully controlled to follow the design. Figure 3-6 shows the steel cage after assembly.

Steel-ply forms were rented from a local company, Form Tech Concrete Forms, Inc. The forms are assembled on a specially constructed, concrete casting bed. After the formwork was erected, form release agent was applied on the inner surface of the forms for easy stripping of formwork. Once the formwork was ready, the steel cages were placed inside, maintaining a concrete clear cover of 1.5” on all sides as shown in Figure 3-7.
Concrete was supplied by a local Ready Mix plant. The slump of the concrete was tested before casting. The specimens were cast directly from the concrete truck. A 1” electric vibrator was used to consolidate the concrete. The concrete surface exposed to air was moist cured with wet burlap and tarp for three days. Figure 3-8 shows a picture during curing of the beams. The forms were stripped after 7 days and kept until reaching 28-day strength.
3.4 Test Setup

The load was applied using a 440 kip capacity hydraulic actuator. Initially the actuators were operated in force control till a load level of 110 kip. Later the actuators were operated in
displacement control. The loading was held at 20 kip intervals to record the cracking using a crack comparator.

The beam was setup on steel plates with a roller in between the plates. The steel roller supports were setup on concrete members which were anchored to the strong floor. The actuator was supported by a steel frame which was also anchored to the strong floor. Figure 3-10 shows the test setup.

![Test Setup](image)

**Figure 3-10: Test Setup**

The test setup was designed to allow each beam to be tested twice, allowing duplication of the test data. Table 3-4 gives the test setup details. The location of the load from the two supports, the effective depth of beams and shear span to depth ratio (a/d) for each group is given in Table 3-4. For Group 1, the load was applied at a distance 76” from
the left support. This configuration allows shear span to depth ratio (a/d) of 3.1. Figure 3-11 shows the Group 1 test setup. After the completion of the first test, the beam was rotated to test the undamaged portion of the beam and the tested portion of the beam had to be cantilevered in order to be able to apply load. The second test on these beams is classified as Group 2. The test setup for Group 2 is shown in Figure 3-12.

<table>
<thead>
<tr>
<th>Group</th>
<th>Target Shear Capacity</th>
<th>Cross - Section (in)</th>
<th>Span (ft)</th>
<th>Load Distance from Left Support (in)</th>
<th>Right Support (in)</th>
<th>Eff. Depth, d (in)</th>
<th>a/d</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$3\sqrt{f'cbd}$</td>
<td>24 x 28</td>
<td>19.0</td>
<td>76</td>
<td>152</td>
<td>25.4</td>
<td>3.1</td>
</tr>
<tr>
<td>2</td>
<td>$3\sqrt{f'cbd}$</td>
<td>24 x 28</td>
<td>13.2</td>
<td>76</td>
<td>82</td>
<td>25.4</td>
<td>3.1</td>
</tr>
<tr>
<td>3</td>
<td>$7\sqrt{f'cbd}$</td>
<td>16 x 22</td>
<td>14.8</td>
<td>51</td>
<td>126</td>
<td>18.0</td>
<td>3.0</td>
</tr>
</tbody>
</table>
Figure 3-11: Group1 Test Setup

Figure 3-12: Group2 Test Setup
For Group 3 beams, the load was applied at a distance of 51” from one end and a distance of 126” from the other end. The cantilevered part of the beam in Groups 3 remains unstressed during load application of the first test. After the first test, the beam was rotated to test the other end. Figure 3-13 shows the Group 3 test setup.

![Figure 3.13: Group 3 test setup](image)

### 3.5 Instrumentation:

All beams were fully instrumented to measure the applied loads on the beams, the deflections crack-widths and strain in steel. In brief, the instrumentation consisted of PI gages, strain gages, and string pots. The same procedure was followed for all the instruments
in terms of calibration. All the instruments were connected to the data acquisition system to record the data and the data acquisition system was the same for all the tests.

One 440 kip capacity actuator was used to measure the applied loads. Two C2A electrical strain gage manufactured by Vishay Micro-Measurements was used to measure the strain in the bottom layer of the longitudinal rebar for all beams. The strain gages were placed on the longitudinal rebar at the location of the applied load. Weldable strain gages were used to measure the strain in stirrups. The location of the main diagonal crack was predicted based on Truss analogy and the weldable strain gage was placed at a location where the diagonal crack intersects the stirrup approximately in the center. Figure 3-14 shows a picture of the strain gage and weldable strain gage.

![Strain gages](image)

**Figure 3-14: Strain gages**
Three rosettes were attached to the face of the beam. For all the beams in Group 3, and beams G1-M100 and G2-M100 the rosette consisted of three 200mm PI gages, placed horizontal, vertical and inclined at 45° angles. For the rest of the beams the rosette consisted of two 200mm PI pages placed horizontally and vertically and one 300 mm PI gage placed inclined at 45° angles. Figure 3-15 shows the rosettes. The rosettes were used to measure the crack widths and strain in stirrups.

Figure 3-15: Rosette Configuration

In addition to the rosettes a crack comparator was used to measure the crack width at different load levels. In addition to the rosettes, six 100mm PI gages were attached to the face
of the beams, to measure the strain in a stirrup. The transverse configuration of 100mm PI-gages is shown in Figure 3-16.

Figure 3-16: Transverse PI gage configuration
4. TEST RESULTS & ANALYSIS

This chapter presents results of the experimental program, and the observed behavior of beams tested to determine the shear behavior of concrete beams reinforced with MMFX shear reinforcement.

4.1 Material Properties

This section provides a summary of concrete and steel mechanical properties used for fabrication of the test specimens based on test results conducted according to ASTM standards.

4.1.1 Concrete

The concrete used in the fabrication of the test specimens was supplied by a local ready-mix plant. Type I Portland cement and 3/8” size aggregates were used in preparing the concrete mix. The material properties was determined using 4 x 8 in. concrete cylinders that were fabricated during each cast and cured under the same environmental conditions as the test beams. The concrete cylinders were tested according to ASTM C39. The average concrete compressive strength on the day of testing is given in Table 4-1. These strength values are used in the analytical phase of this research. The measured average compressive strength of the concrete was higher than the nominal compressive strength used during the design phase of the beams.
Table 4-1: Average Compressive strength of beams on the day of testing

<table>
<thead>
<tr>
<th>Group</th>
<th>ID</th>
<th>Avg. Compressive Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>G1-M0</td>
<td>4468</td>
</tr>
<tr>
<td></td>
<td>G1-C60</td>
<td>4710</td>
</tr>
<tr>
<td></td>
<td>G1-M80</td>
<td>4710</td>
</tr>
<tr>
<td></td>
<td>G1-M100</td>
<td>4953</td>
</tr>
<tr>
<td>2</td>
<td>G2-M0</td>
<td>4468</td>
</tr>
<tr>
<td></td>
<td>G2-C60</td>
<td>4710</td>
</tr>
<tr>
<td></td>
<td>G2-M80</td>
<td>4710</td>
</tr>
<tr>
<td></td>
<td>G2-M100</td>
<td>4953</td>
</tr>
<tr>
<td>3</td>
<td>G3-C0</td>
<td>5356</td>
</tr>
<tr>
<td></td>
<td>G3-M0</td>
<td>5356</td>
</tr>
<tr>
<td></td>
<td>G3-C60</td>
<td>5091</td>
</tr>
<tr>
<td></td>
<td>G3-M80</td>
<td>5238</td>
</tr>
<tr>
<td></td>
<td>G3-M100</td>
<td>5840</td>
</tr>
</tbody>
</table>

4.1.2 Steel

To evaluate the tensile strength of MMFX and Grade 60 steel reinforcement, tension coupons of #3, and #4 bars were tested according to ASTM - A370 specification using a MTS machine. Four coupons for each bar sizes #3 and #4 were taken from the steel used to fabricate the beams and tested. The #11 bars were not tested due to the limited capacity of the
MTS machine. The stress-strain relationship for #11 MMFX bars, used in this report was provided by a research group at University of Texas.

The mechanical properties of the tested grade 60 bars are reported in Table 4-2. The engineering stress-strain relationships of all the #3 and #4 Grade 60 bars are shown in Figure 4-1 and Figure 4-2 respectively. The conventional grade 60 steel used in this experiment had higher yield strength and also it did not exhibit typical yielding plateau for Grade 60 steel. As it did not have a definite yielding plateau the yield strength was measured at a specified offset of 0.2% strain.

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Yield Stress (in ksi)</th>
<th>Ultimate Stress (in ksi)</th>
<th>Young's Modulus (in ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#3</td>
<td>80</td>
<td>103</td>
<td>29,564</td>
</tr>
<tr>
<td>#4</td>
<td>69</td>
<td>105</td>
<td>29,684</td>
</tr>
</tbody>
</table>
Figure 4-1: Engineering Stress-strain Relationship for #3 Grade 60 Bars

Figure 4-2: Engineering Stress-strain Relationship for #4 Grade 60 Bars
The mechanical properties of the tested grade MMFX bars are reported in Table 4-3. The engineering stress-strain characteristics of all the #3 and #4 MMFX are shown in Figure 4-3 and Figure 4-4. In general, the MMFX reinforcing bars exhibit a linear stress-strain relation up to a stress level of approximately 95 ksi for #3 and #4 bars. This linear behavior was followed by an increasingly nonlinear behavior and reduction in the modulus of elasticity up to an ultimate strength of 155 ksi for #3 bars and 160 ksi for #4 bars. As illustrated in Figure 4-3, the stress-strain behavior of the MMFX steel does not exhibit any yielding plateau; therefore the yield strength was measured according to the ASTM-A370 offset method (0.2% offset) at a specified offset of 0.2% (0.002 in/in) strain. After the maximum load was reached in all tests, the diameter of the specimen started to decrease, and the reduction of cross-sectional area was clearly visible in one particular location. This phenomenon is called necking.

### Table 4-3: Mechanical properties of MMFX

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Yield Stress (in ksi)</th>
<th>Ultimate Stress (in ksi)</th>
<th>Youngs Modulus (in ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#3</td>
<td>115</td>
<td>155</td>
<td>29,034</td>
</tr>
<tr>
<td>#4</td>
<td>129</td>
<td>160</td>
<td>29,497</td>
</tr>
<tr>
<td>#11</td>
<td>127</td>
<td>168</td>
<td>29,364</td>
</tr>
</tbody>
</table>
Figure 4-3: Engineering Stress-strain Relationship for #3 MMFX Bars

Figure 4-4: Engineering Stress-strain Relationship for #4 MMFX Bars
As mentioned earlier, the number 11 bars were not tested due to the limited capacity of the MTS machine. The equation used for the stress-strain relationship of the No.11 bars was supplied by the University of Texas as: $f_s = 162 \left(1 - e^{-235\varepsilon}\right)$. 
4.2 **Shear load– Deflection**

The typical measured shear load - deflection behavior up to failure for all tested beams in Group 1, Group 2 and Group 3 is shown in Figure 4-5, Figure 4-6 and Figure 4-7 respectively. The measured deflection is the total deflection, measured directly under the applied load.

![Figure 4-5: Shear Load v/s Deflection for Group 1 beams](image-url)
Figure 4-6: Shear Load v/s Deflection for Group 2 beams

Figure 4-7: Shear Load v/s Deflection for Group 3 beams
The experimental results indicate that the cracking and post cracking stiffnesses for each group of tested beams were identical, regardless of the stirrup spacing and type of reinforcement. The beams without stirrups failed in a very brittle manner at much lower load and significantly less deflection than the beams reinforced with stirrups. The maximum measured deflections at ultimate strength of Group 1, 2, and 3 were approximately 1.0”, 0.8”, and 1.2”, respectively. These relatively small deflections are due to the type of failure which was primarily in shear. When stirrups were added the beams were capable of carrying more load and deflection as well as the failure was more ductile.

The figures also show that despite the lower shear reinforcement ratio for beams reinforced with MMFX stirrups in comparison to beams reinforced with conventional steel stirrups in all the Groups, the beams were capable of sustaining the same loads at similar deflections as the beams reinforced with Grade 60 steel. This behavior is attributed to the utilization of the higher tensile strength of MMFX steel.
4.3 Crack Pattern

The general crack patterns observed for all tested beams were identical. Crack patterns were monitored by holding the loading at various increments throughout the test to mark and photograph the cracks.

The first flexural crack was located near the location of the applied load, where the moment is maximum. The first flexure crack occurred at an applied load of 30 kip, for all the beams. As the load increased the flexural cracks propagated into the compression zone and the number of flexural cracks also increased. In general, all cracks initiated from the bottom and then propagated upward to the top of the beam. Cracks were developed at approximately the location of the stirrups. Therefore, the spacing of cracks was primary controlled by the location of the stirrups. As additional load was applied new flexural cracks began to form towards the support. For beams G1-M0, G2-M0, G3-C0 and G3-M0 i.e. for beams without any transverse reinforcement further increase in load resulted in the formation of a shear crack and sudden failure. The shear crack formation and the failure for these beams are shown in Figure 4-8. On the other hand for beams with transverse reinforcement, the beams were capable of carrying higher load and characterized by the initiation of additional flexure-shear cracks between the applied loads and the supports as shown in Figure 4-9. The beams with transverse reinforcement exhibited a fairly ductile response without an explosive nature.
Figure 4-8: Shear Failure of beams without Transverse reinforcement
As the loading continued, a well defined shear crack formed at the mid-height of the beam, and propagated towards the support and the plates under the applied load in further load cycles. It was observed that the shear crack widened and extended towards the supports at a faster rate compared to the flexure cracks as shown in Figure 4-10.

All the beams, failed due to the crushing of concrete in the nodal zone of the compression strut connecting the nodes at the support and at the applied concentrated loads and a well defined diagonal crack. This failure phenomenon is shown in Figure 4-11.
Figure 4-10: Diagonal-shear cracking

Figure 4-11: Cracking at Failure
Failure of beams, G3-M80 and G3-M100 was associated with a loss of cover concrete. This behavior could be attributed to high stresses in the stirrups and the high compression stresses in the strut, which resulted in crushing of concrete in the strut. Figure 4-12, Figure 4-13 and Figure 4-14 shows the cracking at failure for Group 1, Group2 and Group 3 beams respectively. These figures also include the shear load at failure data.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Failure Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1-C60</td>
<td>134.17k</td>
</tr>
<tr>
<td>G1-M80</td>
<td>123.93k</td>
</tr>
<tr>
<td>G1-M100</td>
<td>112.62k</td>
</tr>
</tbody>
</table>

Figure 4-12: Failure of Group 1 beams
<table>
<thead>
<tr>
<th>G2-C60</th>
<th>Failure = 154.33k</th>
</tr>
</thead>
<tbody>
<tr>
<td>G2-M80</td>
<td>Failure = 135.2k</td>
</tr>
<tr>
<td>G2-M100</td>
<td>Failure = 136.69k</td>
</tr>
</tbody>
</table>

Figure 4-13: Failure of Group 2 beams

<table>
<thead>
<tr>
<th>G3-C60</th>
<th>Failure = 209.06k</th>
</tr>
</thead>
<tbody>
<tr>
<td>G3-M80</td>
<td>Failure = 208.74k</td>
</tr>
<tr>
<td>G3-M100</td>
<td>Failure = 190.39k</td>
</tr>
</tbody>
</table>

Figure 4-14: Failure of Group 3 beams
4.4 Crack width

During testing, crack widths were measured using a crack comparator gage at each load level and using PI gages. The latter method utilizes the geometry of two PI gages in the rosettes in order to record the summation of the shear crack width. The measurements using the crack comparator were comparable to those measured by the PI gages.

The geometry of PI gages as shown in Figure 4-15 was used in order to record the summation of the shear crack width. In the analysis, the vertical and diagonal gages readings were used to calculate the crack width based on the formula in Equation 4-1. The summation of the crack widths is computed using the Equation 4-1, derived from Figure 4-16 (Shehata, 1999).

Figure 4-15: Rosette Configuration
Figure 4-16: Crack Width

\[ \sum w = \left( \sqrt{2} \Delta_D - \Delta_V - 0.5l_g \varepsilon_{ct} \right) \sin \theta + \left( \Delta_V - 0.5l_g \varepsilon_{ct} \right) \cos \theta \tag{Eqn 4-1} \]

where,

\( \Delta_V \) is the measured PI gage readings in the vertical direction

\( \Delta_D \) is the measured PI gage readings in the diagonal direction

\( \theta \) is the measured crack angle to the horizontal beam axis,

\( l_g \) is the gage length of the PI gage, and

\( \varepsilon_{ct} \) is the maximum tensile concrete strain (0.1x10\(^{-3}\)).

The summation of the shear crack widths, \( \sum w \), calculates the width based on all cracks passing through the PI gage. The average width \( \bar{w} \), can then be calculated by dividing the total measurements by the number of cracks.
Currently, the ACI Code neither limits the size of shear cracks nor provides a definitive guideline for the maximum flexural crack width. However, previous editions of the ACI codes and commentary in the current code states that the crack width of 0.016” is assumed to be the maximum flexural crack width based on experimental data. Therefore, this research will assume that at an acceptable service load level the crack widths should be equal to or less than 0.0016”.

The equivalent service load used in the analysis is calculated using 60% of the nominal shear strength resistance from the ACI Code, based on yield strength of 60 ksi for beams G1-C60, G2-C60 and G3-C60, yield strength of 80 ksi for beams G1-M80, G2-M80 and G3-M80 and finally yield strength of 100 ksi for beams G1-M100, G2-M100 and G3-M100. This method of calculating equivalent service load was suggested by Rahal in 2006. The equations to calculate the equivalent service load are given below.

\[
V_n = V_c + V_s \\
V_{\text{service}} = 0.6 \times V_n \\
V_c = 2\sqrt{f_c b_w d} \\
V_s = \frac{A_f f_y d}{s}
\]

As mentioned earlier, Equation 4-1 gives the summation of crack width, to obtain the average crack width at service loads, \( \sum w \) is divided by the number of cracks passing the rosette configuration at service load. Table 4-4 gives the service load levels for each group, the number of cracks recorded at service load for each beam and angle of the crack with
respect to the beam axis. It should be noted that all beams were designed to achieve the same nominal shear capacity using different spacing for the selected yield strength of the steel. Therefore, all beams within each group have the same service load.

Table 4-4: Service Loads

<table>
<thead>
<tr>
<th>Group</th>
<th>ID</th>
<th>b</th>
<th>d</th>
<th>f_c</th>
<th>Stirrup</th>
<th>Spacing</th>
<th>V_c</th>
<th>V_s</th>
<th>V_n</th>
<th>V_n(avg)</th>
<th>V_service</th>
<th>Theta</th>
<th>No. of cracks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>G1-C60</td>
<td>24</td>
<td>25</td>
<td>4710</td>
<td>#3</td>
<td>8.0</td>
<td>84</td>
<td>42</td>
<td>126</td>
<td>128</td>
<td>77</td>
<td>32</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>G1-M80</td>
<td></td>
<td></td>
<td>4710</td>
<td>#3</td>
<td>10.0</td>
<td>84</td>
<td>45</td>
<td>128</td>
<td>128</td>
<td>77</td>
<td>35</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>G1-M100</td>
<td></td>
<td></td>
<td>4953</td>
<td>#3</td>
<td>13.0</td>
<td>86</td>
<td>43</td>
<td>129</td>
<td>128</td>
<td>77</td>
<td>41</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>G2-C60</td>
<td>24</td>
<td>25</td>
<td>4710</td>
<td>#3</td>
<td>8.0</td>
<td>84</td>
<td>42</td>
<td>126</td>
<td>128</td>
<td>77</td>
<td>27</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>G2-M80</td>
<td></td>
<td></td>
<td>4710</td>
<td>#3</td>
<td>10.0</td>
<td>84</td>
<td>45</td>
<td>128</td>
<td>128</td>
<td>77</td>
<td>36</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>G2-M100</td>
<td></td>
<td></td>
<td>4953</td>
<td>#3</td>
<td>13.0</td>
<td>86</td>
<td>43</td>
<td>129</td>
<td>128</td>
<td>77</td>
<td>40</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>G3-C60</td>
<td>16</td>
<td>18</td>
<td>5091</td>
<td>#4</td>
<td>4.5</td>
<td>41</td>
<td>96</td>
<td>137</td>
<td>143</td>
<td>86</td>
<td>47</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>G3-M80</td>
<td></td>
<td></td>
<td>5238</td>
<td>#4</td>
<td>5.5</td>
<td>42</td>
<td>105</td>
<td>146</td>
<td>143</td>
<td>86</td>
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<tr>
<td></td>
<td>G3-M100</td>
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<td></td>
<td>5840</td>
<td>#4</td>
<td>7.0</td>
<td>44</td>
<td>103</td>
<td>147</td>
<td>143</td>
<td>86</td>
<td>49</td>
<td>2</td>
</tr>
</tbody>
</table>

**Group 1**

A measured shear load versus the crack width measured from the PI gages for Group 1 beams is shown in Figure 4-17. The crack width measured from the crack comparator versus the applied shear load is presented in Figure 4-18.
Figure 4-17: PI-gage Crack Width v/s Shear Load for Group 1 beams

Figure 4-18: Crack Comparator Crack Width v/s Shear Load for Group 1 beams
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\[ V_{\text{service}} = 77 \text{ kip} & \quad \text{Applied Load} = 110 \text{ kip} \]

\begin{align*}
\text{G1-C60} & \quad \text{G1-M80} \\
\text{G1-M100} & 
\end{align*}

Figure 4-19: Crack pattern at service load for Group 1 beams
It can be seen from Figure 4-17 and Figure 4-18 that, the measured crack widths from PI-gage and crack comparator are almost the same. Also, it can be seen that all the measured crack width is less than the ACI limit of 0.016”. Due to the high stress induced in the MMFX bar, beam G1-M80 had a higher crack width compared to beam G1-C60. Figure 4-17 show that beam G1-M100 had no cracks at service load. This could be because, beam G1-M100 had a higher concrete compressive strength thus a greater concrete contribution and delaying the formation of the first shear crack. Also, from the crack comparator data it was observed that crack width for beam G1-M100 was 0.0” till a shear load of 86 kip and a crack width of 0.04” was measured at shear load of 86 kip. Figure 4-19 shows number of cracks and crack pattern at service loads for all the beams.

**Group 2**

The shear load versus the crack width measured from PI-gages for Group 2 beams is shown in Figure 4-20. The shear load versus the crack width measured from crack comparator for all Group 2 beams is shown in Figure 4-21. All crack widths measured from both PI-gages and crack comparator were less than the ACI limit of 0.016”. Group 2 beams showed identical behavior as Group 1 beams, except that they had a little higher crack width this is because, during the testing of these beams they already had few cracks from the previous test. Hence, the higher crack width and lesser concrete contribution from the Group 2 beams. Similar to Group 1, the PI-gages and the crack comparator data were almost equal. Measured data indicate that the crack width for beam G2-M80 is greater than the crack width
of beam G2-C60 due to higher induced stresses in the stirrups at the same load level using larger spacing. Figure 4-22 shows the crack pattern and number of cracks at service loads.

Figure 4-20: PI-gage Crack Width v/s Shear Load for Group 2 beams
Figure 4-21: Crack Comparator Crack Width v/s Shear Load for Group 2 beams
\[ V_{\text{service}} = 77 \text{ kip} \text{ & Applied Load} = 150 \text{ kip} \]

Figure 4-22: Crack pattern at service load for Group 2 beams
Group 3

The crack width measured from PI-gages for Group 3 beams is presented in Figure 4-23. The crack width measured from the crack comparator is presented in Figure 4-24. Crack behavior indicates all maximum crack width measured from PI-gage and crack comparator at service load was clearly less than the ACI limit of 0.0016”. The data for all the beams from the crack comparator showed negligibly small crack widths at service load. G3-C60 and G3-M80 have crack width four times under the limit and beam G3-M100 was approximately two times under the limit. Nevertheless, all crack widths are in the acceptable range. The relatively higher crack width of G3-M100 is due to the use of large spacing for the stirrups consequently inducing high stresses at service load. Consequently, test results indicate that even though the MMFX steel stirrups achieve high stresses, the crack widths at service loads are still lesser than the permissible limit. Figure 4-25 shows the number of cracks and crack pattern of Group 3 beams at service loads.
Figure 4-23: PI-gage Crack Width v/s Shear Load for Group 3 beams

Figure 4-24: Crack Comparator Crack Width v/s Shear Load for Group 3 beams
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\[ V_{\text{service}} = 86 \text{ kip} \ & \ Applied \ Load = 130 \text{ kip} \]

Figure 4-25: Crack pattern at service load for Group 3 beams
4.5 Measured Strain in the Stirrups

The strains in the stirrups were measured in three ways:

1. From vertical component of the PI gage rosette configuration shown in Figure 3-13.
2. Using the transverse PI gage configuration shown in Figure 3-14.
3. Using weldable strain gages that were used only in beams G1-M80, G2-M80, G3-C60 and G3-M80.

For all the tests, the three rosettes were used, so that the diagonal crack could capture at least one of the rosettes and this also allowed multiple sets of data to be obtained for each test. In case, the diagonal crack did not pass through any of the rosettes, the data collected from the vertical PI gage were used for analysis. The measured strains based on vertical PI gage were used in the following analysis. The data from the PI gage rosette configuration and weldable strain gages are presented in this section and data from transverse PI gage configuration is presented in Appendix A.

Group 1

Figure 4-26 presents the measured shear load versus transverse strain for Group 1 beams. The figure indicates that the stirrups are stressed only at the load when the PI gage readings start to deviate from the y-axis. This shear load at initial cracking can be approximated as the concrete contribution to shear, \( V_c \). The concrete contribution, \( V_c \), was also estimated from the control specimen, beam G1-M0, which failed at a shear load of 51.4
kip. Figure 4-26 indicates that, beam G1-C60 and G1-M100 has a higher $V_c$, compared with G1-M0 due to the higher concrete compressive strength used for these beams. It is observed that the beams reinforced with MMFX stirrups, G1-M80 and G1-M100 display a linear slope up to strain of 0.01 and then follow the non-linear behavior of MMFX steel as shown in Figure 4-3 and Figure 4-4. It can be seen that at any given load, the strains in all the beams are almost the same i.e. even though beams reinforced with MMFX stirrups have lesser transverse reinforcement ratio compared with beams reinforced with Grade 60 steel, they have approximately similar stains at a given load. This is due to constant elastic modulus of MMFX bars. Within the elastic range, the value is same as conventional steel.

![Figure 4-26: Shear Load v/s Transverse strain for Group 1 beams](image-url)
In beam, G1-C60 stirrups yielded at a strain 0.002 with a corresponding shear load of 72 kip. This proves that the yielding of transverse reinforcement did not cause failure of this beam. From the measured data, it was observed that the longitudinal bars did not yield at failure. The strain in the longitudinal at failure was 0.017. Therefore, the failure was ultimately due to the crushing of the concrete strut. The G1-M80 and G1-M100 beams kept a fairly linear slope up to a strain of 0.01. Because of the high measured strains, the non-linearity towards failure is due to a combination of the stirrups strained within the non-linear range of MMFX, yielding of the longitudinal bars and eventual crushing of the concrete strut.

**Group 2**

Figure 4-27 shows the measured shear load and transverse strain curve for Group 2 beams. In this graph, the curve for beam G2-M100 is not plotted as the main diagonal crack, which caused the failure of the beam occurred on the side which did not have any instrumentation, which is shown on the Figure 4-28.
Figure 4-27: Shear Load v/s Transverse strain for Group 2 beams

Figure 4-28: Failure Crack for beam G2-M100
It can be seen that $V_c$, is approximately 77kip for G2-C60 and 65kip for G2-M80, which is close to the value of 74 kip measured from the control specimen G2-M0. Group 2 exhibits similar behavior to Group 1.

For beam G2-C60, yielding of stirrups occurred at a strain 0.0021 with a corresponding shear load 95 kip. Again, the yielding of stirrup did not cause failure. The strain in the longitudinal bar at failure was 0.0021 which indicate yielding of the longitudinal bars. Therefore, the beams failed due to the yielding of longitudinal bar and crushing of concrete in the strut. G2-M80 exhibits a linear slope until strain of 0.006 and corresponding shear load of 120 kip. For beam G2-M80, the shear load begins to level out, although there is a slight increase in strain. This is from the additional resistance of the MMFX stirrups and/or MMFX longitudinal bars. Increase of strain in the MMFX stirrups in comparison to Grade 60 stirrups is due to the use of larger stirrup spacing of the MMFX steel.

**Group 3:**

Figure 4-29 shows the measured Shear load and Transverse strain for Group 3 beams. For beam G3-C60 it can be seen that, the graph closely follows the stress-strain curve for the #4 bar conventional steel. Failure was due to yielding of longitudinal bars and the crushing of concrete in the compression strut.

For beams G3-M80 and G3-M100, it was observed that, following the formation of the first shear crack the stirrup reached very high strains, without much increase in the load. The stirrup reached a strain of 0.003 at 75kip for both G3-M80 and G3-M100. The Pi-gage
was located exactly on the stirrup, through which the first crack passed. It is believed that, once this stirrup yielded, the adjacent stirrups resisted the applied load. Once all the stirrups in the shear span yielded, additional load is carried by the compression strut and when the concrete at the tip of compression strut crushed, the beam failed. An explosive failure was observed in these two beams. At failure, spalling of the cover concrete was observed during testing. As mentioned in Chapter 3, all the stirrups were bent with 90° hooks. At high stresses these hooks opened up resulting in an explosive failure. This is further discussed in Section 4-6. Further research has to be conducted using stirrups with 135° hooks. Also, the effect of using 90° hooks with longer development lengths should be investigated.

![Graph](image)

**Figure 4-29: Shear Load v/s Transverse strain for Group 3 beams**
4.6 Failure Modes

Shear behavior in reinforced concrete beams is characterized by the occurrence of a composite series of events, including stirrup yielding, stirrup rupture, concrete crushing in the compression zone and concrete crushing in the compression strut. In this section, the key observations of failure modes are presented. Failure of all the beams had nearly identical characteristics. In general, crushing of concrete in the nodal zone of the compression strut was the prevailing mode of failure of test beams. However, four beams G1-M0, G2-M0, G3-C0 and G3-M0 i.e. beams without stirrups failed suddenly in a very brittle manner. A summary of failure loads, deflections, and failure modes is given in Table 4-5.
Table 4-5: Summary of Experimental results

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>Maximum Shear Load</th>
<th>Maximum deflection</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
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<td></td>
<td></td>
<td>(kip)</td>
<td>(in)</td>
<td></td>
</tr>
<tr>
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<td>G1-M0</td>
<td>51.39</td>
<td>0.00</td>
<td>Shear</td>
</tr>
<tr>
<td></td>
<td>G1-C60</td>
<td>134.17</td>
<td>1.11</td>
<td>Shear</td>
</tr>
<tr>
<td></td>
<td>G1-M80</td>
<td>123.93</td>
<td>1.13</td>
<td>Shear</td>
</tr>
<tr>
<td></td>
<td>G1-M100</td>
<td>122.62</td>
<td>1.00</td>
<td>Shear</td>
</tr>
<tr>
<td>2</td>
<td>G2-M0</td>
<td>75.40</td>
<td>0.28</td>
<td>Shear</td>
</tr>
<tr>
<td></td>
<td>G2-C60</td>
<td>154.33</td>
<td>0.76</td>
<td>Shear</td>
</tr>
<tr>
<td></td>
<td>G2-M80</td>
<td>135.20</td>
<td>0.83</td>
<td>Shear</td>
</tr>
<tr>
<td></td>
<td>G2-M100</td>
<td>136.69</td>
<td>0.75</td>
<td>Shear</td>
</tr>
<tr>
<td>3</td>
<td>G3-C0</td>
<td>77.43</td>
<td>0.40</td>
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</tr>
<tr>
<td></td>
<td>G3-C0-R</td>
<td>61.58</td>
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<td>G3-M0-R</td>
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<td>Shear</td>
</tr>
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</tr>
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<td>G3-C60-R</td>
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<td>1.26</td>
<td>Shear</td>
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<tr>
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<td>G3-M80-R</td>
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<td>1.21</td>
<td>Shear</td>
</tr>
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<td>G3-M100</td>
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<td>1.16</td>
<td>Shear</td>
</tr>
<tr>
<td></td>
<td>G3-M100-R</td>
<td>191.94</td>
<td>1.18</td>
<td>Shear</td>
</tr>
</tbody>
</table>

**Group 1**

Beam G1-M0 without any transverse reinforcement failed suddenly, due to the formation of a shear crack. The failure load for this beam is 51.39 kip. Essentially, this failure load indicates the concrete contribution, $V_c$ to the shear capacity of the beams. The concrete contribution to shear capacity of beams is a complex phenomenon, and all the available methods to estimate the concrete contribution give dissimilar results. Therefore the above experimental result provides a best evaluation for the concrete contribution to shear capacity.
For beams reinforced with transverse reinforcement, failure was due to local crushing of concrete in the compression strut. These beams supported considerable additional load after stirrup yielding and before failure.

![Diagram of stress in longitudinal bars for Group 1 beams]

**Figure 4-30: Stress in Longitudinal bars for Group 1 beams**

Figure 4-30 shows a plot of stress in the longitudinal bars versus the applied shear load. From the above figure it can be seen that the longitudinal rebars did not yield and the failure was due to shear-compression resistance of the beams.

It was observed that the beam reinforced with conventional steel, had one major diagonal shear crack, however, beams reinforced with MMFX stirrups had multiple shear
diagonal cracks and clear formation of the diagonal strut. Test results indicate increase in number of cracks and increase in crack width by increasing the shear reinforcement ratio. Figure 4-31 shows the typical failure crack pattern observed during testing. It was also found that the stirrups in beam G1-M100 were ruptured at failure as shown in Figure 4-32.

Figure 4-31: Failure Modes of Group 1 beams.
Group 2:

Beam G2-MO is a control specimen used to estimate the concrete contribution. The beam failed immediately after the formation of shear crack. The failure of this beam can be classified as sudden brittle failure.

The remaining beams had a similar behavior as beams in Group 1. The only difference in Group 1 and Group 2 is the small span used for test setup. Beams G2-C60, G2-M80 and G2-M100 carried an additional load after yielding of the stirrups and failed due to crushing of concrete in the compression strut. The longitudinal bars did not yield in any of these beams.
In the test setup used for Group 2, the load was applied very close to the mid-span of the beam, therefore, as the beam approached failure, the shear diagonal cracks occurred on both sides of the load. For G2-C60 and G2-M80 failure was controlled by the diagonal shear crack in the shorter span which was desired. However, G2-M100 failed due to crushing of concrete in the diagonal crack formed on the longer span as shown in Figure 4-33. The failure was adjacent to the first tested failure part of the beam, therefore, the failure may be affected by bond slipping of the longitudinal bars as evidence by the horizontal crack as shown in Figure 4-33.
Similar to Group 1, it was observed that the beam reinforced with conventional steel, had a single main diagonal shear crack. Beams reinforced with MMFX stirrups had multiple diagonal shear cracks at failure as shown in Figure 4-34, it shows the failure mode observed during testing of the beams.

**Figure 4-33: Failure in beam G2-M100**

![Failure in beam G2-M100](image)
Figure 4-34: Failure Modes of Group 2 beams
Group 3:

In this group, two control specimens G3-C0 and G3-M0 were used to estimate the concrete contribution to nominal shear strength of beams. Beam G3-C0 was reinforced with Grade 60 longitudinal bars and G3-M0 was reinforced with MMFX longitudinal bars. The different longitudinal bars in both the beams did not have any effect on the concrete contribution, $V_c$, to the nominal shear strength. The ultimate failure loads for these beams are presented in Table 4-5. All the beams without transverse reinforcement failed in a brittle manner. The average concrete contribution, $V_c$, for this group is 66 kip.

![Figure 4-35: Stress in Longitudinal bars for Group 3 beams](image-url)
Figure 4-35 shows a plot of stress in the longitudinal bars versus the applied shear load. It can be seen that longitudinal bars in beam G3-C60 yielded at failure. Therefore, the failure was due to the crushing of concrete and yielding of longitudinal bars in beam G3-C60. The longitudinal bars in beam G3-M80 and G3-M100 did not yield, and the failure was due to shear-compression of the beam. All the beams carried an additional load after the yielding of stirrups. Failure of beam G3-M80 and G3-M100 is only due to the crushing of concrete in the compression strut.

Identical to the failure pattern of Group 1 and Group 2 beams, beam G3-C60 reinforced with conventional Grade 60 steel stirrup had one major diagonal shear crack at failure and beams G3-M80 and G3-M100 reinforced with MMFX stirrups had multiple diagonal shear cracks at failure. Another phenomenon observed in this group was that, beams reinforced with MMFX stirrups experienced an explosive failure. The explosive failure is influenced by the opening of MMFX stirrup hooks due to the induced high stresses. This can be seen in Figure 4-36.
Figure 4-36: Failure Mode of Group 3 beams
4.7 Rational Analysis

To verify the observed mode of failure, a rational moment analysis of the tested beams was performed. The analysis compared the measured moment at failure, $M_{exp}$, with the predicted nominal moment capacity of the tested beams using cracked section analysis and moment-curvature analysis. The cracked section analysis utilizes the ACI stress block presented in Figure 4-37 and assumed that the maximum compression concrete strain of 0.003 at extreme compression fiber of the section. The moment-curvature analysis was developed using the Hognestad stress-strain characteristics of concrete, and varying the strain in the concrete from 0 to 0.0038.

The non-linearity of the stress-strain relationship of the MMFX Steel was considered in the analyses to determine the resulting stresses. Table 4-6 shows the ratio of the measured to predicted moment strengths, followed by a column listing the predicted failure mode.

![Figure 4-37: ACI stress block](image)
### Table 4-6: Analysis of beams

<table>
<thead>
<tr>
<th>Group</th>
<th>ID</th>
<th>S (in)</th>
<th>$M_{\text{exp}}$ (kip-in)</th>
<th>Cracked Section</th>
<th>Moment - Curvature</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$M_n$ (kip-in)</td>
<td>$M_{\text{exp}}/M_n$</td>
<td>Failure</td>
</tr>
<tr>
<td>1</td>
<td>G1-C60</td>
<td>8.0</td>
<td>10599</td>
<td>14825</td>
<td>0.71</td>
</tr>
<tr>
<td></td>
<td>G1-M80</td>
<td>10.0</td>
<td>9790</td>
<td>14212</td>
<td>0.69</td>
</tr>
<tr>
<td></td>
<td>G1-M100</td>
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<td>8897</td>
<td>14273</td>
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<tr>
<td>2</td>
<td>G2-C60</td>
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<td>14825</td>
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<td>G2-M80</td>
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<td>14273</td>
<td>0.76</td>
</tr>
<tr>
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<td>11780</td>
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<td>11780</td>
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<td>G3-M80_R</td>
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<td>11782</td>
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<td>G3-M100_R</td>
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<td>10365</td>
<td>12039</td>
<td>0.86</td>
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</tbody>
</table>

In Table 4-6, the failure is defined as “S” for shear or “F” for flexure. If the ratio of $M_{\text{exp}}/M_n$ was greater than one, the failure is defined as flexural failure. Values less than one indicated a shear failure. The ACI moment analysis and moment-curvature agree for each failure mode, as well as the modes observed during testing.
4.8 Predictions

The concrete contribution to shear capacity of beams can be estimated by two means:

1. $V_{c1}$, using the test result from the control specimen without transverse reinforcement and
2. $V_{c2}$, Shear load - Transverse strain relationship at which strain is induced in the stirrups
3. $V_{c3}$, initiation of the first diagonal crack

In this section, the concrete contribution from all the three methods is compared with the predictions from various codes such as ACI, CSA and LRFD. Equations used to calculate the code predictions can be found in Chapter 2. The Table 4-7 lists the predicted ultimate concrete contribution, $V_c$, using ACI, CSA and LRFD code. The ratio of the experimental concrete contribution measured from the control specimen to the predicted ultimate concrete contribution is also presented for each code.
It can be seen that for larger beams i.e. beams in Group 1 and 2, the concrete contribution is overestimated by all the codes. This is due to the size effect. Even though there is experimental data to prove that the concrete contribution to shear capacity of a beam changes with the size of the beam, this fact has not be incorporated in the codes. For smaller sized beams used in group 3, the codes under estimate the concrete contribution.

Also, ACI does not take into consideration the effect of longitudinal steel. If the longitudinal steel is accounted for in ACI, the predictions could be closer to the test results. LRFD provides the closest results when compared with the test data. Since the average of the ratio of measured data to predicted data is 1.06 which is closer to 1 as desired.
**Contribution of the stirrups:**

The steel contribution $V_s$ to the total measured shear carrying capacity during testing, $V_{exp}$, was compared to the predictions value according to ACI-318, CSA, and LRFD codes, to assess the applicability of the code equations in predicting the capacity for High Strength Steel as shear reinforcement. The equations used to calculate the code predictions can be found in Chapter 2. Table 4-8 lists the predicted nominal steel contribution, $V_s$, from each design code as well as the ratio of the experimental to predicted values according to codes. The ratio of the measured/predicted steel contribution for each of the code equations of ACI, CSA and LRFD evaluated is given in Figure 4-27, Figure 4-28 and Figure 4-29 respectively.

Statistical analysis was carried out to find the average, standard deviation, and coefficient of variation based on the shear ratio. The standard deviation indicates the spread of data points relative to the mean, or average. The coefficient of variation is a statistical measure of the dispersion of data points about the mean, and is helpful for comparing data sets with different means. For analysis, it is desirable to have the standard deviation and coefficient of variation as close to zero as possible.
## Table 4-8: Code Comparisons for $V_s$

<table>
<thead>
<tr>
<th>Group</th>
<th>ID</th>
<th>S (in)</th>
<th>$V_{s,exp}$ (kip)</th>
<th>ACI</th>
<th>CSA</th>
<th>LRFD</th>
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<tr>
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<td></td>
<td></td>
<td>$V_s$ (kip)</td>
<td>$V_{s,exp} / V_s$</td>
<td>$V_s$ (kip)</td>
<td>$V_{s,exp} / V_s$</td>
</tr>
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<table>
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</tbody>
</table>
Figure 4-38: Scatter of the Measured / Predicted $V_s$ ratio for ACI 318

Figure 4-39: Scatter of the Measured / Predicted $V_s$ ratio for CSA
From the results presented in Table 4-8 and Figure 4-38, Figure 4-39 and Figure 4-40 it can be seen that the LRFD and CSA equation provide the closest predictions to the resulting steel capacity. The average of the ratios of the measured / predicted stresses from the LRFD equation is 1.01 and CSA is 1.09 compared to 1.42 for the ACI 318. Even though LRFD has a lower average CSA gives a better prediction, because it is over estimating the steel contribution only for tests, i.e. beams G3-M100 and G3-M100-R. But LRFD is over estimating the steel contribution even for beams reinforced with conventional steel, i.e. for beams G3-C60 and G3-C60-R.

The comparison between the measured capacity and the predicted steel capacity by the code equations suggest that the equations proposed by ACI 318 can be used conservatively since it underestimates the effect of stirrups especially when High Strength Steel is used. The
standard deviation and coefficient of variation was similar among each code, showing that each has the relatively same amount of scatter in predicted strength. The ACI is conservative due to the fact it does not adequately account for the contribution of longitudinal rebars which is also high performance steel for the tested beams.

Test results indicate that the ACI-318 can adequately predict the behavior for concrete beams reinforced with MMFX rebars using design strength of 80 ksi and 100ksi. CSA and LRFD adequately predict the shear behavior for rebars using design strength of 80ksi and 100ksi for low shear resistance beams. For higher shear resistance i.e. Group 3 beams, CSA and LFRD under estimate the steel contribution for stress level of 100ksi.
4.9 Response 2000

This section presents the results of a non-linear sectional analysis performed on tested concrete beams reinforced with MMFX steel Response 2000. The computer program RESPONSE 2000 was developed at the University of Toronto by Dr. Evan Bentz as a part of his PhD thesis. This program allows the analysis of reinforced concrete elements subjected to axial load, moment and shear based on the Modified Compression Field Theory. It performs sectional analyses using the stress-strain relationships for the diagonally cracked concrete and the complete stress-strain relationship for the steel reinforcement. Two main assumptions implicit within RESPONSE 2000 is that plane sections remain plane, and that there is no transverse clamping over the beam depth. RESPONSE 2000 can perform three types of analysis: flexural, sectional and member response. Flexural analysis is performed without shear and maximum load and solves for the maximum moment of the beam. Sectional analysis predicts the sectional behavior at any location along the member due to the combined effects of moment and shear. Finally, member response predicts the full member behavior for a given span. In this section, only analysis with shear and member response was used to evaluate the shear behavior of flexural members reinforced with MMFX steel.

All tested beam information such as cross-section, material data, and reinforcement details is input through a graphical interface. In the first step, the title and material properties is input. At this stage only basic material data is input and in the later steps accurate material data can be input. Next, the geometry of the cross section, the number and type of longitudinal bars,
and finally the type and spacing of transverse steel is selected. This creates the entire cross section and shows it graphically on the screen.

In order to obtain accurate analysis, proper material models are needed. Nonlinearity of MMFX steel is an important factor to be considered when trying to model the behavior of reinforced concrete members reinforced with MMFX steel. To be able to understand the mechanical behavior to final failure, it is important that this non-linear behavior of MMFX can be simulated.

For concrete, the program automatically assumes a sophisticated stress-strain curve. The compression softening for normal strength concrete is modeled using the equation proposed by Vecchio - Collins 1986 and tension stiffening is modeled using the equations proposed by Bentz 1999. So, the only value modified here was the compressive strength and aggregate size.

The steel material properties is defined by, elastic modulus, yield strength, strain hardening, rupture strain and ultimate strength. The stress- strain curve for conventional steel and MMFX steel used for analysis was determined by an iterative approach, by changing the parameters mentioned above. Figure 4-41 shows a comparison of the measured stress-strain curve of #3 conventional steel and the model used for the analysis in RESPONSE. Figure 4-42 shows a comparison of the measured stress-strain curve of #3 MMFX steel and the model used for the analysis in RESPONSE.
Figure 4-41: Measured v/s Modeled Stress-Strain curve for #3 Conv. steel

Figure 4-42: Measured v/s Modeled Stress-Strain curve for #3 MMFX steel
Figure 4-40 and Figure 4-41 show a very close correlation between the actual MMFX stress-strain curve and the one Response 2000 used. This fit was created by selecting the ultimate strain to be 0.02, which in reality is much less than the actual. However, because strains at failure are much less than this level, it is an acceptable model. Figure 4-42 below shows a screenshot of the program after a beam’s cross section and material properties have been input.

![Image of a screenshot from Response 2000 showing a beam's cross section](image)

**Figure 4-43: Input Screen shot**

Response 2000 allows moment and shear to be applied to the element. The loading dialogue box is shown below in Figure 4-43. Loading is provided on the left side for a
starting load level or a single load analysis, and on the right for the increments in load. The actual magnitudes of the incremental value are not important. Response 2000 only uses the signs and values relative to each other.

![Figure 4-44: Loading dialogue box](image)

**Sectional Analysis:**

According to Bentz, the key to performing sectional analysis is to choose the proper section. The location chosen for the sectional analysis was taken at a distance equal to the effective depth, $d$, from the edge of the support. This section is just outside of the discontinuity (D-region) and is the most critical section for combined shear and moment effects. The explanation is due to the fact that regions within a length $d$ of a discontinuity (D-region) no longer follow beam theory behavior. As a result, analysis on sections taken inside a D-region will result in very conservative predictions.

After analysis, two control plots are shown one shows shear versus shear-strain plot and other shows the moment curvature plot. Also, a series of nine plots appear showing the
cross section, longitudinal strain, transverse strain, crack diagram, shear strain, shear stress, principal compressive stress, shear on crack, and the principal tensile stress. All the plots show the members behavior just before failure. Response-2000 draws all plots over the depth of the beam, therefore the y-axis for all the plots is the beam depth. A screen shot of this is presented in Figure 4-44. A brief description of the control plots and all the nine plots is given below.

![Figure 4-45: Sectional analysis](image)

**Figure 4-45: Sectional analysis**
Control Plots:

1. **Shear-Shear strain control plots:** The plot shows the maximum predicted shear and shear strain on the section. If, as seen in the Figure 4-44, the plot begins descending with increasing strain, then a shear failure is predicted.

2. **Moment-Curvature control plot:** The plot shows the maximum predicted moment and curvature at the section.

Response-2000 9 Plots:

1. **Cross section:** Darker regions of the cross section indicate uncracked concrete. Longitudinal and transverse reinforcement are drawn in dark red to indicate they are on the yielding plateau and drawn in bright red to indicate strain hardening.

2. **Longitudinal strain:** The linear longitudinal strain confirms indicate the assumption that plane sections remain plane. Maximum and minimum strain values can be obtained from these plots.

3. **Transverse strain:** The transverse strain is not linearly distributed due to local stress-strain conditions. Maximum and minimum strain values can be obtained from these plots.

4. **Crack diagram:** This plot shows the approximate crack pattern and crack width (in inches). For sections where part of the concrete is crushing, the section is redrawn in pink.
5. **Shear strain:** This plot shows the distribution of shear strain in the section. Maximum and minimum strain values can be obtained from these plots.

6. **Shear stress:** Shear stress plot shows the calculated shear stress profile in green and the stress from the strain state in blue. Generally stress from both methods is identical.

7. **Principal compressive stress:** The maximum allowable stress is shown in red, and the applied stress along the section is shown in blue. When the red and blue lines touch, the member fails due to crushing of concrete.

8. **Shear on the crack:** The maximum allowable shear on the crack is shown in red while the applied shear is in blue.

9. **Principal tensile stress:** The maximum allowable stress is shown in red, and the applied stress along the section is shown in blue (psi units). If the red line extends diagonally and intersects the blue line then the section is close to flexural failure.

**Member Analysis:**

A full member analysis will calculate force-deflection relationships for simple beams. Response 2000 requires the length subjected to shear and the type of loading. Once analysis is complete, five plots show up in the main window and two plots in the left window. These include the member crack diagram, curvature distribution; shear strain distribution, deflection, load-max deflection, moment-shear interaction diagram, and the shear-deflection diagram. A screenshot is shown in Figure 4-45. Only the information from the two plots,
Moment-Shear Interaction plot and Shear-Deflection plot in the control chart is used in this analysis. A description of these two plots is given below.

**Figure 4-46: Member analysis**

1. **Moment-Shear Interaction plot:** The vertical axis is the shear (kips) and the horizontal axis is the moment (kip-ft). Response 2000 calculates the outer blue line representing the failure envelope and the finds the largest loading envelop represented by red, which would fit in the failure envelope. If the red line touches the blue line towards the top, then a shear failure is predicted. If it touches on the far right hand side, then a positive flexural failure is predicted.
2. **Shear-Deflection plot**: The vertical axis is the shear (kips) and the horizontal axis is the deflection (in). The corresponding load and deflection is based on the maximum values achieved by the loading envelope in the moment-shear interaction diagram.

Sectional and member analysis were run to predict the behavior of concrete beams. The mode of failure, whether shear or flexure was observed clearly from both the member and section response. The Table 4-10 below shows the results from Response 2000. The nominal shear strength predicted from both section and member response is presented. Results from Response 2000 indicate close correlation with the actual data. It can also be observed that member response gives closer results to experimental data compared to section response. For member response, the average is 1.01 with a standard deviation and coefficient of variation equal to 0.2. One reason Response 2000 is more accurate is because it takes into account the additional resistance provided by the MMFX longitudinal reinforcement. Also, the predictions for beams with MMFX stirrups tend to be as accurate as those using conventional stirrups. This validates the use of the stress-strain curve shown in Figure 4-41.
### Table 4-9: Response results

<table>
<thead>
<tr>
<th>Group</th>
<th>Beam ID</th>
<th>( V_{n,(exp)} )</th>
<th>( V_{n,(response)} ) in kip</th>
<th>( V_{n,(exp)}/V_{n,(response)} )</th>
<th>Predicted Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>G1-C0</td>
<td>51.4</td>
<td>72.0</td>
<td>57.7</td>
<td>0.7</td>
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<tr>
<td></td>
<td>G1-C60</td>
<td>134.2</td>
<td>170.1</td>
<td>164.3</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>G1-M80</td>
<td>123.9</td>
<td>186.9</td>
<td>161.7</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td>G1-M100</td>
<td>122.6</td>
<td>164.6</td>
<td>135.2</td>
<td>0.7</td>
</tr>
<tr>
<td>2</td>
<td>G2-C0</td>
<td>75.4</td>
<td>72.0</td>
<td>57.7</td>
<td>1.0</td>
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<tr>
<td></td>
<td>G2-C60</td>
<td>154.3</td>
<td>170.1</td>
<td>164.3</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td>G2-M80</td>
<td>135.2</td>
<td>186.9</td>
<td>161.7</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td>G2-M100</td>
<td>136.7</td>
<td>164.6</td>
<td>135.2</td>
<td>0.8</td>
</tr>
<tr>
<td>3</td>
<td>G3-C0</td>
<td>77.4</td>
<td>59.6</td>
<td>54.1</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>G3-C0-R</td>
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<td>59.6</td>
<td>54.1</td>
<td>1.0</td>
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<tr>
<td></td>
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<td>54.2</td>
<td>46.2</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>G3-M0-R</td>
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<td>54.2</td>
<td>46.2</td>
<td>1.1</td>
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<tr>
<td></td>
<td>G3-C60</td>
<td>211.4</td>
<td>220.7</td>
<td>207.1</td>
<td>1.0</td>
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<tr>
<td></td>
<td>G3-C60-R</td>
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<td>220.7</td>
<td>207.1</td>
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</tr>
<tr>
<td></td>
<td>G3-M80</td>
<td>212.0</td>
<td>251.4</td>
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<td>0.8</td>
</tr>
<tr>
<td></td>
<td>G3-M80-R</td>
<td>205.5</td>
<td>251.4</td>
<td>229.2</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>G3-M100</td>
<td>188.8</td>
<td>236.9</td>
<td>222.5</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>G3-M100-R</td>
<td>191.9</td>
<td>236.9</td>
<td>222.5</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
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<td></td>
<td></td>
<td>Coefficient of Variation</td>
</tr>
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</table>
The last column gives the predicted failure for each beam. All the beams in the three groups failed in Shear. This mode of failure coincides well with the failure described in Section 4.6 and calculated from moment analysis in Table 4-6. There were three ways to determine the mode of failure in Response 2000. Two methods shear-shear strain diagram and the principal compressive stress from sectional analysis and one method moment-shear diagram from member response. The methods of determining the mode of failure is described below:

1. **Shear-Shear strain control plots:** If the plot begins descending with increasing strain as shown in the Figure 4-46 then the predicted mode of is shear.

   ![Shear-Shear strain plot](image)

   **Figure 4-47: Shear-Shear strain plot**

2. **Principal compressive stress:** For beams with transverse reinforcement, shear failure was characterized by both lines touching, or about to touch, which indicates concrete strut crushing in the section. But, this was not observed in beams without transverse
reinforcement. Figure 4-47 features an example of the allowable and applied principal stresses which indicates a shear failure.

![Principal Compressive Stress Plot](image)

**Figure 4-48: Principal compressive stress plot**

3. **Moment-Shear interaction diagram:** If the red line touches the blue line towards the top, then a shear failure is predicted. Conversely, if it touches on the far right hand side then a positive flexural failure is predicted. This is shown in Figure 4-38.

![Moment Shear Interaction Diagram](image)

**Figure 4-49: Moment Shear interaction diagram**

The three methods of failure for all the beams are presented in the Appendix.
5. CONCLUSIONS

5.1 Summary

This chapter summarizes the research findings of an experimental program completed at North Carolina State University. The research undertaken investigates the shear behavior of new commercially available high performance steel, known as MMFX Steel. The new High Performance Steel is characterized by enhanced corrosion resistance and a significantly higher strength when compared to conventional Grade 60 steel. Use of High Strength reinforcement can lead to significant savings by reducing the material requirements and increasing the service life of flexure members due to its enhanced corrosion resistance.

In this experimental program, eighteen tests were performed on nine large scale rectangular concrete beams. These beams were tested to failure under static loading conditions, in order to evaluate the effectiveness of using MMFX steel as transverse reinforcement in flexural members. The beams were fully instrumented to study the shear behavior of each beam from the beginning to failure. The experimental results were analyzed to describe the shear behavior of beams with MMFX transverse reinforcement. The research included evaluation of the ability of the current codes to predict the shear strength of the MMFX rebars with concrete.

The experimental results and analysis conducted in this research are steered toward the development of design recommendation, to the use of MMFX rebars as shear
reinforcements in flexural members. In addition, results from this experimental program indicate the potential to take advantage of the higher strength offered by MMFX steel bars.

Based on the overall evaluation of each parameter (shear resistance, stirrup spacing and steel type) considered in this investigation several conclusions and recommendations for future work in this field are suggested.

5.2 Conclusions

The following conclusions were made based on significant observations and findings from the testing program:

1. The shear capacity of flexural members can be achieved with lesser amount of MMFX stirrups and lesser MMFX longitudinal reinforcement ratio in comparison to Grade 60 steel. This result is attributed to the higher tensile strength of the MMFX steel in comparison to Grade 60 steel.

2. The beams reinforced with MMFX steel exhibited the same deflections at service load as the beam reinforced with Grade 60 steel. These beams also exhibited almost the same ductility, which is verified by the similar shear load-deflection graphs for beams within each group.

3. Shear crack widths measured for all tested beams reinforced with MMFX steel designed with yield strength of 80 ksi and 100 ksi were within the allowable limit specified by the ACI Code.
4. The ACI, CSA and AASHTO LRFD design codes can conservatively predict the shear behavior of concrete beams reinforced with MMFX steel. The CSA provided the most accurate predictions.

5. ACI 318 does not consider the size effect and the effect of longitudinal reinforcement on the shear strength of members. Therefore, it is more conservative compared to other codes. This factor is most apparent in Group 1 and 2 for the beams reinforced with MMFX steel due to their high tensile capacity.

6. Design stress up to 100 ksi can be for MMFX transverse reinforcement for flexure members without impairing the ultimate load carrying capacity and serviceability. But it is suggested that, the stirrups should have $135^\circ$ hooks, when it is designed for such high stresses.

7. The use of Response 2000, which is based on the Modified Compression Field Theory provides accurate predictions of the overall shear strength of reinforced concrete members containing HP steel.

## 5.3 Future Work

Based on the evaluation of the test results, the following recommendations are given for future work on the use of MMFX as shear reinforcement:

1. Tests should be conducted to verify the use of MMFX as transverse reinforcement in T and I-section concrete beams. The reduced area in the web will serve to greatly decrease shear resistance while the upper flange will maintain an adequate
amount of flexural capacity. This may allow researchers to use lower reinforcement ratios comparable to those actually designed in practice.

2. Test the recommended $135^\circ$ hooks for MMFX stirrups

3. Test the behavior of MMFX steel as ties for concrete columns to explore other applications for bridges, girders and columns.

4. Study the use of MMFX steel in combination with high-strength concrete (HSC).
REFERENCES


2. ACI Committee 318 (2008). *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (318R-08).* Farmington Hills: American Concrete Institute.


References


APPENDICES
6. APPENDIX A: Test Data

6.1 Longitudinal bar Details

Bent Bar Details:

<table>
<thead>
<tr>
<th>Steel</th>
<th>Size</th>
<th>Length</th>
<th>Inside bend radius, R</th>
</tr>
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<tbody>
<tr>
<td>MMFX</td>
<td>#11</td>
<td>1' - 1”</td>
<td>21' - 7”</td>
</tr>
<tr>
<td>CONV.</td>
<td>#11</td>
<td>1' - 1”</td>
<td>21' - 7”</td>
</tr>
</tbody>
</table>
Appendix A

Test Data

Straight Bar Details:

<table>
<thead>
<tr>
<th>Steel</th>
<th>Size</th>
<th>Length</th>
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<tr>
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<td>21' - 9&quot;</td>
</tr>
<tr>
<td></td>
<td>#10</td>
<td>21' - 9&quot;</td>
</tr>
<tr>
<td></td>
<td>#11</td>
<td>21' - 2&quot;</td>
</tr>
<tr>
<td>CONV.</td>
<td>#9</td>
<td>21' - 9&quot;</td>
</tr>
<tr>
<td></td>
<td>#10</td>
<td>21' - 9&quot;</td>
</tr>
<tr>
<td></td>
<td>#11</td>
<td>21' - 2&quot;</td>
</tr>
</tbody>
</table>

6.2 Stirrup details

<table>
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<th>Steel</th>
<th>Detail</th>
<th>Size</th>
<th>Dimension</th>
</tr>
</thead>
<tbody>
<tr>
<td>MMFX</td>
<td>A</td>
<td>#3</td>
<td>1'-9&quot; x 2'-1&quot;</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>#4</td>
<td>1'-1&quot; x 1'-7&quot;</td>
</tr>
<tr>
<td>CONV.</td>
<td>A</td>
<td>#3</td>
<td>1'-9&quot; x 2'-1&quot;</td>
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<td></td>
<td>B</td>
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<td>1'-1&quot; x 1'-7&quot;</td>
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## Appendix A

### Test Data

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<td></td>
<td>2'-1&quot;</td>
</tr>
<tr>
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</tr>
<tr>
<td></td>
<td></td>
<td>2.25&quot;</td>
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<tr>
<td>#4</td>
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<td>1'-6&quot;</td>
</tr>
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<td></td>
<td>1'-1&quot;</td>
</tr>
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</table>
6.3 Beam Loading picture during testing

Figure 6-1: Loading till Failure for Beam G1-M0
Figure 6-2: Loading till Failure for Beam G1-C60
Figure 6-3: Loading till Failure for Beam G1-M80
Figure 6-4: Loading till Failure for Beam G1-M100
Figure 6-5: Loading till Failure for Beam G2-M0
Figure 6-6: Loading till Failure for Beam G2-C60
Figure 6-7: Loading till Failure for Beam G2-M80
Figure 6-8: Loading till Failure for Beam G2-M100
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<th>MMFX stirrups</th>
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<tr>
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**Figure 6-9: Loading till Failure for Beam G3-C0 and G3-M0**
Figure 6-10: Loading till Failure for Beam G3-C60
Figure 6-11: Loading till Failure for Beam G3-M80
7. **APPENDIX B: Response**

7.1 **Response 2000 results**

---

**Figure 7-1: Response Results for beam G1-M0 and G2-M0**

---
Figure 7-2: Response Results for beam G1-C60 and G2-C60

Figure 7-3: Response Results for beam G1-M80 and G2-M80
Figure 7-4: Response Results for beam G1-M10 and G2-M10

Figure 7-5: Response Results for beam G3-C0
Figure 7-6: Response Results for beam G3-M0

Figure 7-7: Response Results for beam G3-C60
Appendix B

Response

152

Control : $V_{Gxy}$

Principal Compressive Stress

top

bot

Control : $M-V$

Control : $P-\Delta$

Figure 7-8: Response Results for beam G3-M80

Control : $V_{Gxy}$

Principal Compressive Stress

top

bot

Control : $M-V$

Control : $P-\Delta$

Figure 7-9: Response Results for beam G3-M100