CYCLIC RESPONSE OF CONCRETE BEAMS
REINFORCED WITH ULTRAHIGH STRENGTH STEEL

A Dissertation in
Architectural Engineering
by
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Submitted in Partial Fulfillment
of the Requirements
for the Degree of

Doctor of Philosophy

August 2011
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ABSTRACT

Seismic applications of concrete members reinforced with ultrahigh strength steel bars (yield strength greater than 80 ksi) have been limited by U.S. building codes since the introduction of Strength Design in 1963. The limitation has been primarily due to the paucity of experimental data. This investigation aimed to provide benchmark data for studying the cyclic response of concrete beams reinforced with steel bars having yield strengths approaching 100 ksi.

Seven specimens were subjected to large transverse displacement reversals: three specimens were reinforced longitudinally with conventional steel bars (Grade 60) and four with ultrahigh strength steel bars (Grade 97). All transverse reinforcement was Grade 60. Other experimental variables were: volume fraction of steel-hooked fibers (0 or 1.5%), spacing of transverse reinforcement (d/4 or d/2), and ratio of compression-to-tension longitudinal reinforcement (ρ’/ρ = 0.5 or 1). The nominal concrete compressive strength was 6000 psi.

Test beams reinforced with Grade 97 bars had drift ratio capacities in excess of 10%, comparable to the deformation capacities of similar beams reinforced with conventional bars. Numerical models were developed for the calculation of moment-curvature and force-displacement relations. The calculated values showed good correlation with the measured data.

For beams with nearly identical strength, the measured yield displacement of beams reinforced with Grade 97 bars was about 25% greater than in beams reinforced with Grade 60 bars. However, nonlinear seismic analyses of single-degree-of-freedom systems indicate that the displacement calculated for systems representing the beams with Grade 97 bars are, on average, less than 10% larger than the displacement calculated for systems representing the beams with Grade 60 bars.

The evidence presented shows that the use of ultrahigh strength steel (Grade 97) as longitudinal reinforcement is a viable option for earthquake-resistant construction.
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ACKNOWLEDGMENTS

Acknowledging the advisor at first has been an old tradition in academia, and probably this is one of the few times, that I am happy to bind to an old tradition. However, my intention to thank Dr. Lepage is not only out of respect for a tradition. In addition to Dr. Lepage, I would also like to thank the members of my doctoral committee, Prof. Memari, Prof. Parfitt, and Prof. Lopez. I have been benefited from their support.

I would also like to express my gratitude to Dr. Geschwindner, Dr. Anumba, Dr. Boothby, and Dr. Linzell for their support during my Ph.D. studies.

The author would like to thank fellow student Hiroki Ota for his valuable help in casting specimens and fabricating experimental equipment. I would also like to express my appreciation to Daniel Fura and Paul Kremer for their technical assistance in the Civil Engineering and Architectural Engineering Laboratories.

I would also like to thank my student fellows and friends; Gaby Issa-El-Khoury, Elisabetta Pistone, and Mohammad Heidarinejad.

Nothing but the least, I would like to thank my family. I am grateful for all types of supports I received from them and without which I would have never been able to finish my studies. I would like to thank my parents, who have always been supportive even from thousands miles away. Additionally, I would like to express my gratitude to my uncle and aunt in State College for their kindness and hospitality.

Primary financial support for this project was provided by the Department of Architectural Engineering and College of Engineering, allocated by Dr. Lepage. This financial support is highly acknowledged and appreciated. Partial support was provided by SAS Stressteel, Inc.
CHAPTER 1

INTRODUCTION

1.1 Problem Statement

For many years, the design of reinforced concrete structures for seismic applications in the U.S. has been dominated by the use of deformed steel bars with a specified yield strength, $f_y$, of 60 ksi. Using steel reinforcement with a higher yield strength would introduce several benefits to the architectural, engineering, and construction industry. Not only would it allow for members with reduced amounts of reinforcement, saving on material, shipping, and placement costs, but it would also help to overcome steel congestion, and facilitate concrete placement. This would lead to better quality of construction, improved durability, and shorter construction times. One of the primary obstacles to designing and constructing concrete members reinforced with ultrahigh strength steel ($f_y$ greater than 80 ksi) is the paucity of experimental data.

The limitations on yield strength of reinforcing steel can be traced back to the requirements of the 1956 ACI Code (ACI 318-56) where the maximum yield stress of 60 ksi was specified. This limit was increased to 75 ksi in 1963 (ACI 318-63) provided that full-scale beam tests were carried out to demonstrate that the average crack width at service load did not exceed 0.015 in. Later, the 1971 version (ACI 318-71) relaxed the limit on $f_y$ to 80 ksi to continue accommodating the highest strength covered by contemporary ASTM standards.

Beginning more than a decade ago, with high-grade steel bars becoming more popular around the world, research has been conducted in the area of high strength reinforcement, with special emphasis on studying the effect of high strength steel transverse reinforcement on concrete confinement. ACI 318-08 (2008) introduced new limits of up to 100 ksi on the yield
strength of reinforcement, but only for the design of spirals and ties as confinement reinforcement in compression members.

Steel with yield strength above 80 ksi is currently produced commercially and often referred to as Advanced High Strength Steel (AHSS) or Ultrahigh Strength Steel (UHSS) (ISSI, 2006; ASTM A1011, 2010). Nowadays, with the availability of ultrahigh strength steel (e.g., MMFX Grade 100, or Grade 120 conforming to ASTM A1035, 2007), applications as main longitudinal bars in concrete members are on the horizon. Although there is high interest to use ultrahigh strength bars, experimental evidence is needed on the behavior of concrete members reinforced with UHSS bars, especially when used as main longitudinal reinforcement and when subjected to cyclic loading beyond yielding.

The deformability of conventional concrete both in tension and compression needs to be enhanced in order to make effective use of unprecedented high yield strengths and elongations of newly developed UHSS reinforcing bars. The deformation capacity of UHSS-reinforced concrete members may be enhanced by the addition of closely spaced transverse reinforcement and/or by the use of high performance fiber reinforced concrete (HPFRC). Throughout this study, RC refers to reinforced concrete applications without the addition of fibers, while HPFRC refers to applications of fiber reinforced concrete characterized by strain-hardening behavior after first cracking and exhibiting multiple cracks at relatively high strain levels (Naaman, 1987).

There is no evidence in the U.S., to date, of any structure that has been built or rehabilitated using UHSS as primary longitudinal reinforcement, in RC or HPFRC, taking advantage of yield strengths in excess of 80 ksi for compression and tension stresses. This study was undertaken to address this gap.
1.2 Objectives and Scope

The main objective of this research was to study the behavior of concrete beams, longitudinally-reinforced with UHSS bars, subjected to cyclic loading. A discussion of the observed response during testing of seven specimens is included. The discussion focuses on the measured force-displacement hysteresis curves, strains in the longitudinal and transverse reinforcement, crack widths, and stiffness reductions.

A numerical model was developed to simulate the behavior of the specimens and comparisons were made between the calculated and measured response. The idealized stress-strain curves of the concrete and reinforcing steel were based on measured properties. The model was formulated to account for the enhanced deformability of concrete due to the presence of transverse reinforcement and/or high performance steel-hooked fibers. Strain-hardening effects of the reinforcing bars are also included in the analyses.

The main experimental variables were: yield strength of longitudinal reinforcement (Grade 60 or Grade 97); spacing of transverse reinforcement ($d/2$ or $d/4$); volume fraction of steel-hooked fibers (0% for RC or 1.5% for HPFRC); and ratio of compression-to-tension reinforcement ($\rho'/\rho = 0.5$ or 1). The controlled parameters included: shear span-to-effective depth ratio of 3; nominal concrete compressive strength of 6000 psi; and Grade-60 transverse reinforcement.

1.3 Organization

Background on the use of high strength steel in RC members is presented in Chapter 2, including seismic applications. A review of recent developments on the use of HPFRC is also
presented in Chapter 2 with a description of theoretical models representing the mechanical behavior of RC and HPFRC.

Chapter 3, in conjunction with Appendix A, describes the test specimens, testing facility, and instrumentation. Details are given for the fabrication of the specimens, measured material properties, loading protocol, and testing procedure.

Experimental results are presented in Chapter 4 using extensive graphs and tables that illustrate the behavior of the test specimens. The measured data include force-displacement response, crack widths, and strains of longitudinal and transverse reinforcement. Behavior of each specimen is discussed and comparisons are made between specimens.

A numerical model is formulated in Chapter 5 to simulate the experimental results. A computer program based on moment-curvature analysis is used to simulate the force-displacement response of the test specimens and comparisons are made between the calculated and measured response. Nonlinear seismic analyses of single-degree-of-freedom systems are performed to compare the displacement response associated with changes in hysteresis due to the use of Grade-97 or Grade-60 reinforcement.

A summary of the research program and concluding remarks on the use of ultrahigh strength steel as concrete reinforcement are presented in Chapter 6. Two sets of conclusions are given; one related to the use of Grade-97 bars in reinforced concrete without fibers (case of RC), the other to the use of Grade-97 bars in high performance fiber reinforced concrete (case of HPFRC).
CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

Previous experimental studies on the application of high strength steel are reviewed in this chapter. In addition, experimental investigations conducted on fiber reinforced concrete (FRC), as related to the scope of this study, are briefly summarized.

2.2 High Strength and Ultrahigh Strength Steel Reinforcement

Early investigations on concrete members reinforced with high strength steel bars trace back to the extensive column investigation by Richart and Brown (1934), where the effect of high strength steel with yield strength up to 96 ksi (665 MPa) was studied. In 1936, the ACI Building Code (ACI 501-36T), based on allowable stress design, changed the allowable compression stress in the vertical column reinforcement to forty percent of the yield point but not to exceed 30 ksi, instead of the modular ratio $n$ times the allowable concrete stress that was adopted in previous versions of the ACI Code. A stress of 30 ksi was allowed on bars with a yield point of 75 ksi (Hognestad, 1961).

Germundsson (1941) outlined the architectural and economic advantages of using Grade 75 reinforcement. However, it was not until the late 1950s that practical applications of high strength steel in reinforced concrete began in the U.S. In 1959, ASTM standards for high strength steel bars (Table 2.1) were published (Hognestad, 1961).

A vast laboratory investigation (Hognestad, 1961) started in 1955 at PCA laboratories to address four performance requirements for concrete members reinforced with high strength steel:
(1) Adequate load-carrying capacity;

(2) Limited formation of cracks in the concrete;

(3) Adequate rigidity or limited deflections;

(4) Adequate durability.

The high strength bars in the PCA experiments ranged in yield strengths from 55 to 120 ksi (379 to 828 MPa) but mainly less than 90 ksi (621 MPa). The findings of the PCA investigations were presented in a series of reports in the 1960s (Hognestad, 1961; Hognestad, 1962; Gaston and Hognestad, 1962; Kaar and Mattock, 1963; Pfister and Mattock, 1963; Pfister and Hognestad, 1964; Kaar and Hognestad, 1965, Kaar, 1966).

To address the second requirement, multiple tests were performed that led to the following major findings (Hognestad, 1962):

- Initial flexural cracking formed in a random manner and at an irregular spacing. They began at a relatively low steel stress of 10 to 18 ksi (69 to 124 MPa).

- All major cracks were usually formed at a steel stress of 30 ksi (207 MPa). At higher stresses the existing cracks widened, but new cracks seldom formed.

- Crack width was proportional to steel stress.

- Size of deformed bars did not have a significant effect on crack width.

- No clear relationship between crack width and reinforcement ratio was found.

- Crack width was essentially independent of concrete compressive strength, beam width, and beam depth.

- Crack width was strongly influenced by the thickness of concrete cover.

In Part 3 of the PCA reports, the results of testing a full scale 60-ft (18.3-m) long roof girder were presented (Gaston and Hognestad, 1962). The main tension reinforcement had a specified yield strength of 70 ksi (483 MPa). The supplied steel bars had an average yield strength of 83 ksi (572 MPa). The compression reinforcement, stirrups, and side bars (skin
reinforcement) were intermediate grade steel having an average yield point of 41 ksi (283 MPa).

The specified compressive strength of concrete was 5000 psi (34 MPa), but at the time of the final
test, the actual concrete strength was 9900 psi (68 MPa). The testing was carried out in two main
stages:

(1) A sustained load test in which the dead load alone was applied for 82 days.

(2) A test with incremental loading until collapse.

The behavior of the girder was satisfactory and the ultimate load carried by the member
was 21% greater than the calculated load using actual material strengths. The full-scale tests
confirmed the following:

- Ductile behavior may be realized even when high strength materials are used. The
  ultimate deflection was approximately equal to one thirtieth of the span.

- At failure, the concrete strain in the compression flange was approximately 0.003, while
  the tensile steel strain exceeded 0.01.

- Additional strain measurements showed that the distribution of strain across a section was
  practically linear.

- If deflections cannot be safely accommodated without impairing the service function for
  which the members are designed, member depths should be increased or high strength
  reinforcement should not be used.

One of the main concerns in using high strength reinforcement is the potential for
crushing of concrete before developing the yield strength of reinforcing bars. This concern is
expressed in the commentary to Section 1505 of ACI 318-63:

“This section provides limitations on the use of high strength steels to assure
safety and satisfactory performance. High strength steels frequently have a strain
at yield strength or yield point in excess of 0.003 assumed for the concrete at
ultimate. The requirements of Section 1505(a) are to adjust to this condition.

The maximum stress in tension of 60,000 psi without test is to control cracking.
The absolute maximum is specified as 75,000 psi to agree with the present
ASTM specifications and as a safeguard until there is adequate experience with high stresses.”

The commentary on Section 1508 of ACI 318-63 states:

“When the design yield point of tension reinforcement exceeds 60,000 psi, detailing for crack control becomes even more important. Entirely acceptable structures have been built, particularly in Sweden, with a design yield strength approaching 100,000 psi but more design criteria for crack control and considerable American practical experience with 60,000 psi yield strength tension reinforcement are needed before higher yield strengths are approved for general use. The Code, therefore limits tension reinforcement to 60,000 psi yield strength, unless special full-scale tests are made. The crack width criteria are not too difficult to meet by proper attention to reinforcing details.”

Todeschini et al. (1964) investigated the use of high strength reinforcement in concrete columns both experimentally and theoretically with special emphasis on the effect of eccentricity of loading, concrete strength, and percentage of reinforcement. All reinforcing bars used by Todeschini et al. were made of high strength billet steel meeting the requirements of ASTM A431 with specified yield strength of 75 ksi (517 MPa). Their findings include:

- All failures were compression failures due to crushing of the concrete.
- Buckling of the compression steel was secondary and followed concrete crushing.
- The theoretical analysis was based on the concept that the load carried by a column is the sum of that carried by the concrete and the portion carried by the steel. At strains higher than \( \varepsilon_0 \) (strain at peak concrete stress) the load carried by the concrete would decrease. A critical strain is attained after \( \varepsilon_0 \) where the rate of decrease of the load carried by the concrete is larger than the rate of increase of the load carried by the steel. The strain at which this takes place is assigned the theoretical ultimate strain, \( \varepsilon_u \). Using the above method of analysis, the calculated load carrying capacity of the columns was accurate.
- The ultimate compressive strain in the concrete increases with increase in eccentricity (in columns loaded eccentrically).
• An increase in concrete strength decreases the ratio $\epsilon_u/\epsilon_0$, while the increase in percentage of longitudinal reinforcement produces an increase in this ratio.

• It appears unlikely that concrete compressive strains of 0.006 can be obtained. In some few cases, strains of 0.004 may be expected, with maximum strains of 0.003 occurring most frequently.

• The experimental results show that strains somewhat above 0.003 for the axially loaded specimens, and at least between 0.003 and 0.004 for the eccentrically loaded ones, can be developed with high strength steel bars. For the type of reinforcing steels used, this represents a utilization of steel strengths in compression of up to 70 to 80 ksi (483 to 552 MPa).

• It is anticipated that even for reinforcing steels with flat yield plateaus up to approximately 90 ksi (621 MPa), the yield strength in compression can be developed. Otani et al. (1996) summarized the results of more than a hundred beam tests carried out using high strength reinforced concrete beams in the 1980s and early 1990s. The data were obtained from multiple sources: Proceedings of Japan Concrete Institute (JCI), Summaries of Technical Papers of Annual Meeting of Architectural Institute of Japan (AIJ), Annual Reports of the New RC project, and reports of research institutes of Japanese construction companies. Otani et al. made reference to more than 25 laboratory reports. Most of these reports are in Japanese without translation. The concrete strength in the above mentioned tests ranged from 4400 to 17400 psi (30 to 120 MPa). The longitudinal bars had yield strengths ranging from 58 to 174 ksi (400 to 1200 MPa). The ultrahigh strength transverse reinforcement, which is commonly used in Japan, had a yield strength ranging from 87 to 188 ksi (600 to 1300 MPa). The data reported by Otani et al. suggest that high strength steel is not uncommon in the Japanese reinforced concrete industry.
Watanabe et al. (1995) used an analytical approach (moment-curvature) to show the behavior of columns reinforced with a combination of bars of different grades, yield strengths of 58, 116, and 174 ksi (400, 800, and 1200 MPa). They concluded that the combined use of high strength longitudinal bars with bars of conventional strength (Grade 60) improves the moment-curvature relationships of column sections under several types of loadings, such as uniaxial, biaxial, monotonic, and cyclic. However, they pointed out that practical problems need to be addressed, such as possible splitting bond failure and elastoplastic buckling of the ultrahigh strength longitudinal bars. They have also mentioned that there were very few data on buckling of ultrahigh strength reinforcement.

In the last decade more research has been devoted to investigate the seismic response of concrete members with high strength steel, but the majority has focused on the effect of ultrahigh strength transverse reinforcement. The transverse reinforcement in concrete members not only resists the shear forces and helps prevent rebar buckling, but it also provides confinement for the concrete core which leads to improvements in the ultimate deformation capacity and ductility of concrete members.

Wight and Sozen (1973) stated that the rate of strength decay in columns under shear reversals was a function of axial load, percentage of transverse reinforcement, and lateral deflection. Based on the test results of twelve reinforced concrete columns, they found that yielding of the transverse reinforcement usually corresponded to the start of the reduction in shear capacity for most of the test specimens. If the transverse reinforcement did not yield, the column sustained its strength under cyclic loading, otherwise a permanent lengthening of the hoop occurred in subsequent cycles and the column strength decayed. This suggests that the use of high strength steel as transverse reinforcement would be very effective in delaying strength decay.

Lin and Lee (2001) studied the effect of stirrup strength on beam ductility under monotonic loading. The yield strength of the stirrups varied between 53 and 80 ksi (363 and 554
MPa). They found that the confining effect of transverse reinforcement did not exert an apparent influence on the maximum strength of the beam because the confining effect was not evident before spalling of the concrete cover. The test data showed that the relationships between curvature ductility and stirrup strength were almost linear.

Pessiki et al. (2001) developed a procedure for the design and analysis of high strength spiral reinforcement based on large-scale axial compression tests. The spiral yield strength in these experiments ranged from 78 to 195 ksi (538 to 1340 MPa). The proposed design procedure is based upon a calculated usable spiral stress rather than the nominal yield stress of the spiral reinforcement. The procedure satisfactorily predicted the behavior of the test specimens with usable spiral stress values up to 110 ksi (758 MPa).

In a research carried out in New Zealand, Bing et al. (1994) tested square and circular columns under axial load with normal and ultrahigh strength transverse steel, having yield strength of 190 ksi (1320 MPa). The use of ultrahigh strength transverse reinforcement generally enhanced the axial strength and ultimate compression strain of the confined concrete. However, ductile behavior was severely limited in the plastic hinge regions for the specimens tested in combined flexural and axial loading for cases where the axial load approached 0.6 $f'_{c}A_g$.

Muguruma et al. have reported that the use of ultrahigh strength transverse reinforcement with yield strength of 127 ksi (873 MPa) improved the flexural ductility of square columns tested under reversed cyclic lateral loads with constant axial load in the range of 0.3 to 0.5 $f'_{c}A_g$. They also reported a delay in buckling of the conventional (Grade 60) longitudinal bars for the specimens reinforced with ultrahigh strength transverse reinforcement.

The prevention of buckling of the longitudinal bars in specimens with ultrahigh strength transverse reinforcement, with yield strengths of 198 ksi (1370 MPa), has also been reported by Sato et al. (1993).
Azizinamini and Saatcioglu (1998) have noted that increasing the spacing of transverse reinforcement due to higher yield strength may lead to buckling of conventional longitudinal bars.

Budek et al. (2002) published the results of nine column tests which were transversely reinforced with prestressing strands of $f_{pu} = 250$ ksi (1723 MPa). Their summary of results from previous tests indicates:

- Under axial compression, ultrahigh strength transverse reinforcement enhances concrete compression strength.
- Axial strain rate did not have a significant effect on the performance of columns confined with ultrahigh strength transverse reinforcement.
- Full utilization of the capacity of ultrahigh strength transverse reinforcement may be limited by reduced dilation of high strength concrete.
- The higher yield strain of ultrahigh strength transverse reinforcement generally delayed buckling of the longitudinal bars.
- There is a lack of experimental data addressing the lateral load performance of spirally reinforced circular columns using ultrahigh strength transverse reinforcement.

The experimental data obtained by Budek et al. suggest that using a conservative value of $0.6f_{pu}$ for maximum allowable tensile stress in the transverse reinforcement provided adequate shear strength while significantly reducing steel congestion. The transverse reinforcement in the plastic hinge regions generally remained in the elastic range. They indicated that the use of conventional (Grade 60) transverse reinforcement would have yielded in the plastic hinge regions. Where prestressing strands are used as transverse reinforcement, Budek et al. recommended a spiral pitch of four times the longitudinal bar diameter to prevent the buckling of conventional longitudinal reinforcement.

Restrepo et al. (2006) studied the seismic performance of bridge columns using longitudinal reinforcement with a yield strength (0.2% offset) of 94 ksi (648 MPa) and transverse
reinforcement with a yield strength of 135 ksi (931 MPa). They found that the columns designed with ultrahigh strength steel result in smaller initial stiffness and greater yield displacements than columns designed with Grade 60 reinforcing steel. They indicated that a reduction in the amount of reinforcement in proportion to the yield strength ratios might not be achievable in components subjected to plastic deformations (due to limited ductility). They mentioned that an advantage of using higher strength transverse reinforcement is the effectiveness to restrain the longitudinal reinforcement and avoid buckling.

Seliem et al. (2008) tested three full-scale bridge decks to examine the flexural limit state behavior, including the mode of failure. Two of the bridge decks were constructed with the same reinforcement ratio but one used ultrahigh strength steel (Grade 120) and the other used conventional Grade 60 rebar. The third bridge deck was reinforced with ultrahigh strength steel (Grade 120) with a reinforcement ratio of two thirds of the others. The reduction in the amount of steel reinforcement was based on a nearly linear-elastic behavior up to a rebar stress of 90 ksi (621 MPa). The test results led to the following conclusions:

- The primary mode of failure for the three bridge decks was punching shear.
- The three bridge decks behaved as uncracked sections under service load level. Therefore, using one-third less ultrahigh strength steel should not alter the serviceability behavior of concrete bridge decks.
- The bridge deck reinforced with one third less ultrahigh strength steel provided nearly the same ultimate load-carrying capacity as the one reinforced with Grade-60 steel.

The above review indicates that most studies on the seismic applications of ultrahigh strength steel in reinforced concrete have focused on its use as transverse reinforcement. The recent investigation by Restrepo et al. (2006) did focus on the cyclic response of concrete members reinforced with ultrahigh strength steel bars as longitudinal reinforcement but it was
limited to circular columns and its response was severely impaired by the premature failure of welded hoops at the weld.

### 2.3 Stress-Strain Relationship of Confined and Unconfined Concrete

To predict and understand the behavior of reinforced concrete members, it is important to have a reliable mathematical model for the stress-strain curve of concrete in tension and especially in compression.

Different models for the stress-strain behavior of concrete in compression have been proposed by many researchers. For unconfined concrete, the model proposed by Hognestad (1951) is adopted in this study as presented in Chapter 5.

Park and Paulay (1975) recommended an idealized stress-strain curve of concrete in tension using a straight line up to the tensile strength of concrete, with the modulus of elasticity assumed to be the same as in compression.

The compressive stress-strain curve of confined concrete differs from unconfined concrete depending on the amount and spacing of transverse reinforcement. Transverse reinforcement provides passive confinement to concrete in compression. At low levels of compressive stress in the concrete, the transverse steel barely affects the axial response of concrete and therefore the stress-strain curve is similar to unconfined concrete. As mentioned by Park and Paulay (1975), the shape of the stress-strain curve for confined concrete depends on many variables:

- The ratio of the volume of transverse reinforcement to the volume of the concrete core;
- The yield strength of the transverse reinforcement;
- The ratio of the spacing of the transverse reinforcement to the dimension of the concrete core;
• For cases where transverse reinforcement is provided by rectangular hoops, the ratio of the length of the hoop leg to the diameter of the hoop reinforcement;
• The amount and size of the longitudinal reinforcement;
• The compressive strength of concrete;
• The rate of loading.

Kent and Park (1971) proposed the stress-strain curve shown in Figure 2.1 where the ascending branch AB is defined similar to the model by Hognestad (1951) and the descending branch BC is defined using:

\[ BC: \text{ for } 0.002 \leq \epsilon_c \leq \epsilon_{20c}, \quad f_c' = f_c' [1 - Z(\epsilon_c - 0.002)] \]  \hspace{1cm} (2.1)

Where

\[ Z = \frac{0.5}{\epsilon_{50u} + \epsilon_{50h} - 0.002} \]  \hspace{1cm} (2.2)

\[ \epsilon_{50u} = \frac{3 + 0.002f_c'}{f_c' - 1000} \]  \hspace{1cm} (2.3)

\[ \epsilon_{50h} = \frac{3}{4} \rho_s \frac{b''}{S_h} \]  \hspace{1cm} (2.4)

\( b'' \) = width of confined core measured to outside of hoops;
\( S_h \) = spacing of hoops;
\( \rho_s \) = ratio of the volume of transverse reinforcement to the volume of the concrete core measured to outside of hoops.

Later, a modified Kent and Park model (Scott et al., 1982) incorporated the factor \( k f_c' \) in Eq. (2.1), where \( k \) was defined in terms of the volumetric reinforcement ratio, \( \rho_s \).

Roy and Sozen (1964) suggested a simplified model for confined concrete based on experimental data from columns with rectangular hoops. The Roy and Sozen model is adopted in this study as described in Chapter 5.
A comprehensive stress-strain model for confined concrete for different confinement parameters was proposed by Mander et al. (1988a and 1988b). They stated that confinement is improved if

- The transverse reinforcement is placed at relatively close spacing.
- Additional supplementary overlapping hoops or cross ties with several legs crossing the section are included.
- The longitudinal bars are well distributed around the perimeter.
- The volume of transverse reinforcement to the volume of the concrete core is increased.
- The yield strength of the transverse reinforcement is increased.
- Spirals or circular hoops are used instead of rectangular hoops.

The effect of confinement on high strength concrete has been investigated by Liu et al. (2000). They found that the strength of columns with high strength concrete is affected by spalling of the cover. The concrete core is not capable of carrying the increased loads after the loss of cover for the cases where a larger cover was used. Paultre et al. (1996) and Foster and Attard (2001) showed that by adding steel fibers to the high strength concrete matrix, cover spalling can be effectively prevented.

All of the above clearly conveys that transverse reinforcement plays a major role in extending the useful concrete compressive strain.

### 2.4 Fiber-Reinforced and High-Performance Fiber-Reinforced Concrete

A strategy to improve the usable compressive strain of concrete is to add fibers. The concept of adding fibers as reinforcement into concrete is not new. The use of horsehair as fibers to reinforce mud bricks can be traced back to ancient Egypt (Li, 2002). ACI 116R, Cement and
Concrete Terminology, has defined Fiber Reinforced Concrete, as concrete containing dispersed randomly oriented fibers (Zollo, 1997).

The idea of fiber-reinforced concrete (FRC) was first brought to the attention of academic researchers in the early 1960s by Romualdi and Batson (1963) and Romualdi and Mandel (1964). Throughout the present study, the term FRC is adopted to refer to fiber reinforced cementitious composites regardless of the type of fiber and size of aggregates.

To analyze and design the structural elements made of FRC, the stress-strain curve of this material is needed. In general, adding fibers to a plain concrete matrix has a minor effect on its behavior before cracking of the matrix. However, the post-cracking behavior is highly affected and improved by the presence of fibers. The mechanical properties of FRC are highly dependent on the type and amount of fibers (Zollo, 1997; Li, 2002). Before summarizing the findings of relevant research on FRC, the following generic terminology is adopted:

- **Fiber volume fraction** ($V_f$): the ratio of volume of fibers to the volume of FRC.
- **Fiber aspect ratio** ($l/d$): fiber length divided by fiber equivalent diameter.
- **Fiber reinforcing index** ($RI$): the product of the above two terms, $V_f$ and $l/d$.

Fanella and Naaman (1985) carried out a comprehensive experimental program to study the effects of steel, glass, and polypropylene fibers on the compressive stress-strain curves of mortars. Their findings include:

- The ascending portion of the stress-strain curve is only slightly modified by the addition of fibers, whereas the descending portion is modified significantly. Higher fiber content produces a less steep descending portion, leading to higher ductility and toughness of the material.

- Except for the case of steel fibers, adding fibers to a concrete matrix did not improve its compressive strength. However, the strain at the peak compressive stress is increased by
the presence of any fiber type. Compressive strength with steel fibers increased up to 15%.

- For the same fiber volume fraction, steel fibers led to higher matrix toughness than glass or polypropylene fibers. Steel fibers exhibited pullout at increasing strains while glass and polypropylene fibers generally failed in tension before pulling out.

- The strain at the peak stress of fiber reinforced concrete was found to vary linearly with the volume fraction of fibers. For the case of steel fibers, this strain was also found to vary linearly with the fiber reinforcing index.

Different types of fibers are currently used to reinforce plain concrete matrices. Based on Fanella and Naaman (1985), steel fibers are very efficient in enhancing the mechanical behavior of concrete mixes.

Craig (1987) studied the behavior of singly and doubly reinforced concrete beams with and without fibers. The FRC beams used crimped steel fibers, with fiber aspect ratios between 60 and 100, at volume fractions below 2%. The test data from 13 beams support the following benefits of FRC:

- Cracking is more uniformly dispersed;
- The stiffness is increased;
- The moment capacity is increased;
- The ductility is increased;
- The ultimate concrete compressive strain is increased;
- Concrete spalling and buckling of compression steel are prevented.

Swamy and Al-Noori (1975) speculated that the presence of steel fibers enables the use of high strength steel in structural members, allowing both crack width and deflection to be controlled within acceptable limits:
“Since crack width is directly related to the steel stress, it is obvious that for a given crack width, a higher steel stress could be permitted in the longitudinal reinforcement, and the use of fiber concrete would therefore permit the use of steels with higher characteristic strength than at present.”

Tests by Soroushian, and Bayasi (1987) corroborated that randomly distributed fibers also effectively improve the toughness, impact resistance, and tensile strength of concrete. Like in the case of compression, fiber type, aspect ratio, and volume fraction are determining factors. These factors dictate the average number of fibers per unit cross sectional area:

\[
N_1 = \alpha \frac{V_f}{A_f}
\]  

(2.5)

Where, \(N_1\) is the average number of fibers per unit area; \(A_f\) is the cross sectional area of an individual steel fiber; and \(\alpha\) is the orientation factor (Soroushian and Lee, 1990). The orientation factor is a constant ranging from 0.41 to 0.82. If the fibers are randomly distributed in an infinite 3D space (without boundary effects) the orientation factor is equal to \(\frac{4}{\pi^2} = 0.41\). The number and type of fibers affect the mechanical behavior of FRC.

In addition to the number and type of fibers, the shape of the fibers is another determining factor. Soroushian and Bayasi (1991) stated that for steel fibers, hooked fibers are more effective than straight ones for enhancing the flexural and compressive behavior of concrete.

Fibers also improve the bond stress between reinforcing bars and concrete. The higher the fiber volume fraction, the higher the bond strength and the higher the area under the load vs. slip response of the reinforcing bars (Hota and Naaman, 1997).

Li (2002) presented an overview of fiber applications in cementitious composites for the civil engineering industry. Li observed that most applications are nonstructural:
“One of the major drawbacks in many current FRC applications is that the development of the FRC is often decoupled from the design of the concrete element.”

Recently, more effort has been devoted to use fibers for solving structural problems with High Performance Fiber Reinforced Cementitious Composite members. The term High Performance Concrete is often used to describe concrete with special properties, such as ease of placement and consolidation, high strength, durability, and other desired properties (MacGregor and Wight, 2005). The term High Performance Fiber Reinforced Concrete (HPFRC) refers to a class of FRC characterized by a tensile stress-strain curve with strain-hardening after first cracking and multiple cracking up to relatively high strains, see Figure 2.2 (Naaman, 1987). To achieve quasi strain-hardening and multiple cracking in tension, a minimum volume fraction of fibers (referred to as critical volume fraction) should be used so that the maximum post-cracking stress ($\sigma_{pc}$) is larger than the stress at first cracking ($\sigma_{cc}$).

Using the expression for $\sigma_{pc}$ and $\sigma_{cc}$ suggested by Naaman (1987) leads to a closed-form solution for the critical volume fraction of fibers (Liao et al., 2006):

$$V_{f\text{ critical-tension}} = \frac{1}{1 + \frac{\tau l}{\sigma_{mu} d} (\lambda_1 \lambda_2 \lambda_3 + \alpha_1 \alpha_2)}$$

(2.6)

where:

$V_f$ = volume fraction of fibers;

$l$ = length of fiber;

$d$ = fiber diameter;

$\sigma_{mu}$ = tensile strength of matrix;

$\tau$ = average bond strength at the fiber-matrix interface;

$\alpha_1$ = coefficient representing the fraction of bond mobilized at first matrix cracking;

$\alpha_2$ = efficiency factor of fiber orientation in the uncracked state of the composite;
\(\lambda_1\) = expected value of the ratio of the fiber pull-out length to length of fiber;
\(\lambda_2\) = efficiency factor of orientation in the cracked state;
\(\lambda_3\) = group reduction factor for the number of fibers pulling out per unit area.

Li (2002) identified the advantageous properties of HPFRC for seismic applications:

- High compression strain capacity avoids loss of integrity by crushing.
- Low tensile first cracking strength initiates damage within the plastic hinge.
- High shear and spall resistances avoid loss of integrity by diagonal fractures.
- Enhanced mechanisms increase the inelastic energy dissipation.

Several researchers have studied the behavior of members cast with HPFRC during the last decade. Fischer and Li (2002) investigated the effect of matrix ductility on the cyclic response of HPFRC flexural members with conventional steel rebar. As previously mentioned, transverse reinforcement provides confinement to concrete members and consequently adds ductility to the member and also contributes to resist the shear forces. Fischer and Li examined the idea of using HPFRC as replacement for transverse reinforcement. They found that the shear resistance provided by stirrups for confinement was redundant in the HPFRC specimens. It is important to note that the shear stresses in the specimens by Fischer and Li did not exceed \(2.5 \sqrt{f'_{c}}\) psi \((0.21 \sqrt{f'_{c}}\) MPa) a relatively low value compared to those accepted in modern engineering applications. Fischer and Li concluded that the damage in reinforced HPFRC members was dominated by flexural cracking with stable inelastic deformations of steel bars without buckling. HPFRC members were found to be more damage tolerant than RC members.

Parra-Montesinos et al. (2005) evaluated HPFRC as a means to eliminate the need for transverse reinforcement in beam-column connections subjected to cyclic loading. The fiber used in their experiments was polyethylene with a volume fraction of 1.5\%. Their HPFRC specimens were not provided with special transverse reinforcement in the beam plastic hinge regions. They
concluded that the ACI 318 Code limitations on joint shear stresses could be safely applied to HPFRC beam-column connections without transverse reinforcement. There were no signs of bar buckling throughout the tests.

Parra-Montesinos and Chompreda (2007) found that a shear stress of $3.5\sqrt{f_{ct}}$, psi (0.29 $\sqrt{f_{ct}}$, MPa) is a reasonable lower bound for determining the concrete contributions to shear strength ($V_c$) in HPFRC members, even at displacements of about 9% the shear span.

ACI 318-08 (2008) is the first version of ACI 318 Code that allows the use of steel fibers to replace the use of transverse reinforcement for resisting shear forces in non-seismic applications. A strength of $2\sqrt{f_{ct}}$, psi (0.17 $\sqrt{f_{ct}}$, MPa) is assigned to FRC as long as:

- The compressive strength of concrete is limited to a maximum of 6000 psi (41 MPa).
- Beam depth does not exceed 24 in. (0.61 m).
- Steel fibers are hooked or crimped with a length-to-diameter ratio between 50 and 100, and conform to ASTM A820 (2006).
- Steel fiber-reinforced concrete beams are tested per ASTM C1609 (2006) and satisfy all of the following:
  a) Flexural strength at a midspan deflection of 1/300 of span length is greater than or equal to 90% of the first-peak strength.
  b) Flexural strength at midspan deflection of 1/150 of span length is greater than or equal to 75% of the first-peak strength.
  c) Fiber volume fraction exceeds 0.75%.
CHAPTER 3
EXPERIMENTAL PROGRAM

3.1 Introduction

Seven specimens were tested to study the cyclic response of concrete beams reinforced with ultrahigh strength steel bars. This Chapter describes the specimens, the test procedure, and the instrumentation used.

3.2 Specimen Description

The specimens consisted of two beams connected to a central stub (Figure 3.1). Each beam element was intended to represent a cantilever beam with the central stub acting as the base of the cantilever. The specimens were 88-in. long, consisting of two 36-in. long beams and a 16-in. long central stub. The beams were supported by rollers located at 24-in. from the face of the central stub. The cross section of the beams was identical in all specimens, 16-in. wide by 10-in. deep, with an effective depth of 8 in. Reinforcing details of each specimen are shown in Figure 3.2.

All beams were proportioned to have nearly identical flexural strengths. Beam dimensions were primarily chosen so that the maximum induced shear stress \(\frac{V}{bd}\) approached \(6\sqrt{f'_c}\) (psi), a value on the high end of what is permitted in Chapter 21 of ACI 318-08 (2008) for moment-frame beams developing \(1.25f'_c\) in the longitudinal reinforcement. Transverse reinforcement was provided to resist shear, to confine the concrete core, to help prevent buckling.
of the longitudinal reinforcement, and to enhance the bond between the concrete and the longitudinal bars.

The main experimental variables were: the yield strength of the longitudinal reinforcement (Grade 60 or Grade 97); the volume fraction of steel-hooked fibers (0% for RC specimens or 1.5% for HPFRC specimens); spacing of transverse reinforcement (d/4 or d/2); and ratio of compression-to-tension longitudinal reinforcement ($\rho'/\rho = 0.5$ or 1). The controlled parameters included: nominal compressive strength of concrete ($f'_c = 6000$ psi); nominal yield strength of transverse reinforcement ($f_y = 60$ ksi); and shear span-to-effective depth ratio ($a/d = 3$). Material properties and as-built dimensions are presented in Appendix A.

Transverse reinforcement consisted of #3 rectangular hoops in accordance with Chapter 21 of ACI 318-08 (2008). The HPFRC specimens had 1.5% volume fraction of steel-hooked fibers (Dramix RC-80/30-BP) added to the same concrete mix that was used to cast the non-fiber RC specimens. The fibers, commercially available and manufactured by Bekaert Corporation (http://www.bekaert.com), had a length of 1.2 in. and a diameter of 0.015 in., for an aspect ratio of 80. The tensile strength of these fibers is 330 ksi (2300 MPa).

Specimens are designated with four to five characters that refer to the experimental variables. The first character identifies the type of longitudinal reinforcement, where “C” is used for conventional steel with $f_y = 60$ ksi and “U” is used for ultrahigh strength steel with $f_y = 97$ ksi. The second character refers to the type of steel used as transverse reinforcement. The third character indicates the spacing of the transverse reinforcement, 4 for $d/4$ and 2 for $d/2$. The fourth character is either “F” for HPFRC or “X” for RC. The fifth character, $\$, is only used to identify specimens with $\rho'/\rho$ of 0.5. Table 3.1 shows a summary description of the specimens. Note how every specimen has one or more related specimens that differ in only one parameter (e.g., CC4-X relates to UC4-X and CC4-X$\$).
3.3 Test Procedure

A detailed description of the experimental procedure is presented in Appendix A. All specimens were loaded at the central stub, see Figure 3.1 and Figure 3.3. The target displacement history was the same for each specimen. The loading protocol consisted of increasing cycles from drift ratios of 0.15% to drift ratios of 5% in accordance with the recommendations of FEMA 461 (Table 3.2 and Figure 3.4), where drift ratio is defined as the ratio of displacement to shear span. The applied displacement history is meant to simulate the effects induced by strong ground motions.

During cycles below 1% drift ratio, the beams on both sides of a specimen experienced similar drift ratios. As the longitudinal reinforcement yielded, the central stub rotated and the drift ratio of one beam became larger than the drift ratio of the adjacent beam (refer to Figure A.33). The beam with the larger drift ratio was used to control the loading protocol.

After completing the second cycle to 5% drift, the specimen was subjected to a monotonically increasing deformation until failure (defined by a reduction in strength of 20% or more from the measured peak). The applied cyclic displacements induced nonlinear force-displacement response in the test beams, simulating the effects induced by strong ground motions.

3.4 Instrumentation

A detailed description of the instrumentation used in the experiments is presented in Appendix A. The specimens were instrumented to measure displacements, forces, and strains.

To calculate the net deflection of each beam, displacements were measured at 8 locations using linear potentiometers (Figure 3.5). All 8 displacements were required to calculate and
cancel out the rigid body translations and rotations of the specimen during cyclic response. Thereafter, the net deflections were used to calculate drift ratios for each beam (see Section A.4). Deformations in the beams and the central stub were also mapped using electronic Whittemore gages (Figure A.36) that were designed and built for this experimental program. Two gages of different sizes (6 in. and 8.5 in.) were used to monitor deformations between anchor points laid on a 6-in. by 6-in. grid (Figure A.35).

The force applied to each specimen was measured using a custom-made load cell. The shaft connecting the Hydraulic Enerpac Cylinder (the actuator) was instrumented with three strain gages to perform as a load cell (Figure 3.6). The procedure used to design and build this custom-made load cell is described in Section A.4.

Longitudinal and transverse reinforcement were also instrumented with strain gages. These strain gages were attached to the reinforcement cage at strategic locations as shown in Figure 3.7. Types of gages and their installation procedure are described in Section A.4.
CHAPTER 4
EXPERIMENTAL RESULTS

4.1 Introduction

The measured experimental data for the test specimens are presented in this chapter. The data include the measured shear, stub displacement and rotation, strain in longitudinal reinforcement, and strain in transverse hoops. In addition to the directly measured data, other parameters derived based on the measured data are presented here. Cracks were marked and the maximum widths of flexural cracks and shear cracks were measured and recorded at the peak of each load cycle. Photos at critical stages of loading showing the appearance of the specimens are also included.

Displacement readings, described in Chapter 3 and Appendix A, led to the calculation of drift ratios for each beam. The value of shear in each beam was determined by dividing the total applied force by two. Strains on the reinforcing cage were measured at different locations by means of strain gages (see Appendix A).

4.2 General Observations

A description of the behavior of each of the test specimen is included in this section, for specimen description see Table 3.1. General observations are made at specific drift ratios with supporting photographs. Table 4.1 summarizes the general quantitative observations related to drift ratio and shear. None of the specimens failed in shear. The footnotes in Table 4.1 suggest that using the traditional model in ACI 318 Code, where \( V_n = V_c + V_s \), transverse reinforcement
spaced at $d/4$ was sufficient to carry the applied shear assuming that the shear strength attributed to the concrete, $V_c$, was zero. The three HPFRC specimens (UC2-F, CC2-F, and UC2-F$\var$), with hoops spaced at $d/2$, relied on $V_c \geq 3\sqrt{f'c} (\text{psi}) b d$.

Specimen #1; CC4-X: this specimen was designed according to the requirements of Chapter 21 of ACI 318-08 (2008) for a special moment frame beam. The specimen was built using conventional Grade-60 reinforcement. It was capable of completing the loading protocol. In the final push (monotonic loading), CC4-X reached a drift ratio of 15% without failure (defined in Chapter 3).

Figure 4.1 shows the specimen at different stages of testing. At early stages (below 0.4% drift ratio) only flexural cracks were observed. At the start of the first cycle to 0.4% drift ratio small inclined cracks were observed. At a drift ratio of 1%, small signs of cover spalling were observed. However, the amount of spalling was relatively minor until a drift ratio of 4%. In the first cycle of 2% drift, flexural cracks in the north beam became too wide to measure with a crack comparator (more than 0.06 in.) in the region of maximum moment. It was similar for the south beam after the specimen reached a drift ratio of 3%. At a drift ratio of 4%, shear cracks became too wide on both beams and crack width measurement by a crack comparator was discontinued. In spite of wide flexural and shear cracks, the north beam exceeded 15% drift ratio without failure (Figure 4.2). At the end of the test, the specimen was unloaded and the loose cover concrete was removed (Figure 4.3). The extent of shear cracks in the concrete core was visible after cover removal.

Specimen #2; UC4-X: the second specimen was very similar to the first one, except that the longitudinal bars were replaced by ultrahigh strength steel (Grade 97). This specimen successfully completed the loading protocol. It was also capable of resisting the applied force during the final push without failure up to a drift ratio exceeding 15%.
Figure 4.4 shows the specimen at different stages of testing. After the first cycle, hairline flexural cracks were observed. Hairline inclined cracks also showed up soon after. At a drift ratio of 1%, small signs of cover spalling were observed. However, the amount of spalling was small until 3% drift ratio. In the first cycle of 3% drift, flexural cracks became too wide to be measured by a crack comparator (more than 0.06 in.) in the region of maximum moment in both the north and south beams. Inclined shear cracks started to grow at drift ratios in excess of 1.5% and soon became too wide to be measured by a crack comparator at a drift ratio of 4%. Considerable concrete cover spalled off during the cycles targeting 4% and 5% drift ratios. In spite of wide flexural cracks, the north beam exceeded 15% drift ratio without failure. Due to saturation of the sensors measuring drift, values after a drift ratio of 15% became unreliable. However, shortly after exceeding 15% drift, the north beam failed due to fracture of the tensile reinforcement (Figure 4.5). At the end of the test, the specimen was unloaded and the loose concrete cover was removed (Figure 4.6). The extent of shear cracks in the concrete core was visible after cover removal.

Specimen #3; UC4-F: the third specimen was a replica of the second specimen except that it was made of HPFRC. This test was designed to investigate the effect of fibers on the behavior of beams. Specimen UC4-F successfully completed the loading protocol. It was also capable of resisting the applied force during the final push without failure up to a drift ratio of 12%.

Figure 4.7 shows the specimen at different stages of testing. Hairline flexural cracks were detected at the third cycle of loading (0.2% drift ratio). The crack widths remained almost unchanged up to cycles of 0.6% drift ratio. However, additional cracks were developed in each subsequent cycle. Inclined cracks were not developed before the second cycle of 0.6% drift ratio. The inclined crack widths were almost constant up to the first cycle of 2% drift ratio. At a drift ratio of 1.5%, small signs of cover spalling were observed. However, the amount of spalling was
insignificant even up to the failure point (12% drift ratio). In the first cycle of 3% drift, flexural cracks became too wide to be measured by a crack comparator (more than 0.06 in.) in the vicinity of maximum bending moment in both the north and south beams. Thereafter, one of the main flexural cracks grew in size (length and width) in subsequent cycles, while the development of new cracks was not noticeable. During cycles exceeding 3% drift, the relative movement of the two parts of concrete adjacent to the dominant crack was observed at shear values less than 20 kips (sliding shear). Inclined shear cracks started to grow after the domination of the aforementioned single flexural crack. In spite of the wide flexural crack, the north beam reached 12% drift ratio where fracture of a longitudinal bar occurred. At the end of test, the specimen was unloaded but the concrete cover could not be removed to expose the core (Figure 4.8). It was noticeable that damage of the specimen (especially the north beam) was concentrated at the face of the stub.

Specimen #4; UC2-F: the fourth specimen was similar to the third one except for the spacing of the hoops which was increased from \(d/4\) to \(d/2\). This test examined the shear capacity and confining capability of fibers. Specimen UC2-F successfully completed the loading protocol. It was also capable of resisting the applied force during the final push without failure up to a drift ratio of 11%.

Figure 4.9 shows the specimen at different stages of testing. Hairline flexural cracks were detected at the third cycle of loading (0.2% drift ratio). The crack widths remained almost unchanged up to cycles of 0.6% drift ratio. However, more cracks developed in subsequent cycles. Inclined cracks were developed after the second cycle of 0.8% drift ratio. The inclined crack widths were almost unchanged up to the first cycle of 2% drift ratio. At a drift ratio of 1.5%, small signs of cover spalling were observed. However, the amount of spalling was minor even after reaching 11% drift in the final push. In the first cycle of 2% drift ratio, flexural cracks exceeded 0.06 in. in the south beam. Cracks exceeded 0.06 in. in the north beam during the cycles
of 4% drift ratio. Thereafter, one of the main flexural cracks grew in size (length and width) and new cracks were not identified. During cycles exceeding 3% drift, there was a relative movement of the concrete adjacent to the dominant crack at shear values below 20 kips. Inclined shear cracks started to grow after the localization of the aforementioned flexural crack. In spite of the presence of a wide flexural crack, the south beam reached 11% drift ratio at which the fracture of a longitudinal bar occurred. At the end of the test, the specimen was unloaded but the concrete cover could not be removed (Figure 4.10). The damaged part of the specimen (especially the south beam) was concentrated at the face of the stub.

Specimen #5; CC2-F: this specimen was similar to Specimen CC4-X except that due to presence of fibers the spacing of hoops was increased from \( d/4 \) to \( d/2 \). This specimen successfully completed the loading protocol. During the final push the specimen reached a drift ratio of 15% without failure.

Figure 4.11 shows the specimen at different stages of testing. Chronologically this specimen was the first one tested. This specimen was setup and ready to be tested in the Civil Engineering Laboratory, but limitation of the test apparatus moved the testing program to the Architectural Engineering (AE) Laboratory. Because the specimen was painted at least twice with a lime-based paint, the layers of paint made it difficult to measure the crack widths; therefore, the cracks were marked but not measured. The specimen completed cycles up to 1.5% drift ratio without any major damage. At this stage, small signs of cover spalling were observed. However, the amount of spalling was small even up to the end of the test. After reaching 15% drift without failure, the specimen was unloaded and the concrete cover could not be removed (Figure 4.12). Spalling concentrated around the dominant flexural cracks near the face of stub.

Specimen #6; CC4-X$: this specimen was similar to Specimen CC4-X except that instead of four longitudinal bars at top and bottom of the section, it had four bars at bottom and two bars at top. This specimen successfully completed the loading protocol. In the final push, CC4-X$
reached a drift ratio of 15% without failure. During the final push the bottom bars were in tension.

Figure 4.13 shows the specimen at different stages of testing. At early stages (up to 0.4% drift ratio) only flexural cracks were observed. At the start of the first cycle of 0.6% drift ratio, small inclined cracks were observed. At a drift ratio of 1% small signs of cover spalling were observed. However, the amount of spalling was minor until a drift ratio of 4%. In the second cycle of 2% drift ratio, flexural cracks in the south beam became too wide to be measured by a crack comparator (more than 0.06 in.) in the region of maximum moment. A similar observation for the north beam occurred during the cycle of 4% drift ratio. In spite of wide flexural and shear cracks, the south beam exceeded 15% drift ratio during the final push (Figure 4.14). At the end of test, the specimen was unloaded and the loose concrete cover was removed (Figure 4.15). The extent of shear crack in the concrete core was visible after cover removal.

Specimen #7; UC2-FS: this specimen was similar to Specimen UC4-F except that it had two reinforcing bars at top and four bars at bottom. This specimen completed the loading protocol and during the final push, failure occurred at a drift ratio of 11% due to fracture of the bottom longitudinal reinforcement.

Figure 4.16 shows the specimen at different stages of testing. Hairline flexural cracks were detected at the third cycle of loading (0.2% drift ratio). The crack widths remained almost unchanged up to cycles of 0.6% drift ratio. However, more cracks were developed in the following cycles. Inclined cracks were developed after the second cycle at a drift ratio of 0.8%. The inclined crack widths were nearly unchanged up to the first cycle of 3% drift ratio. At a drift ratio of 1.5%, small signs of cover spalling were observed. However, the amount of spalling was minor even up to failure occurring at a drift ratio of 11%. In the first cycle of 2% drift ratio, flexural cracks in the south beam became too wide to be measured by a crack comparator (more than 0.06 in.) in the region of maximum moment. Thereafter, one of the main flexural cracks
grew in size (length and width) in subsequent cycles and new cracks were not identified. Upon load reversal, at values of shear below 20 kips, there was a relative movement of the concrete adjacent to the dominant flexural crack. Inclined shear cracks continued to grow even after the presence of the dominant flexural crack. During the final push, the south beam failed at a drift ratio of 11% due to fracture of a longitudinal bar. At the end of test, the specimen was unloaded but concrete cover could not be removed (Figure 4.17). The main damaged part of the specimen (especially the south beam) was a small region close to the face of stub.

4.3 Measured Response

4.3.1 Measured Shear vs. Drift Ratio Curves

Figure 4.18 to Figure 4.27 show the measured shear force vs. drift (rotation) of the test specimens for the cyclic part of the tests. For a detailed description of drift calculation see Appendix A. These figures show the shear-drift relationship of both south and north beams. Usually at the beginning of each test, both beams behaved similarly and the average drift ratio of the two beams was used to define the target drift ratio of the loading protocol. However, in some cases, after a few cycles in the nonlinear range of response, one beam deteriorated more than the other. In such cases, the loading protocol was controlled by the drift of the softer beam. In all cases, except for Specimen UC2-F, (see Figure 4.24) the softer beam was properly identified after yielding of the longitudinal bars (usually the first cycle to 1.5% drift ratio for UC specimens). For Specimen UC2-F, before the first cycle at 2% drift ratio, the north beam was controlling but thereafter, the south beam controlled. Observe in Figure 4.24 that when the north beam attained 2% drift ratio, the south beam reached 4% drift ratio.
At the end of the cyclic portion of the loading protocol, each specimen was monotonically loaded until failure of the specimen or to the limit of the test apparatus. Figure 4.32 to Figure 4.38 show the monotonic part of test only for the controlling beam. These figures clearly show that even for specimens reinforced with ultrahigh strength steel, a high level of ductility is attained. The overall behavior of each specimen is summarized in terms of the envelope curve (all-positive quadrant) of shear vs. drift ratio for the controlling beams in Figure 4.39 to Figure 4.45. To draw the envelope curve the value of shear associated with each step of the loading protocol (Table 3.2 and Figure 3.4) is plotted against the corresponding nominal drift ratio.

### 4.3.2 Measured Moment-Curvature Relationships

The measured moment-curvature relationships along the instrumented length of each beam are presented in this section. The strain values obtained for the longitudinal bars at the top and bottom reinforcement layers were added algebraically and then divided by the vertical distance between the layers to obtain the curvature at the location of longitudinal strain gages (Figure 3.7).

Figure 4.46 to Figure 4.50 show the measured curvature against the corresponding value of moment. All graphs are drawn using the same scale to facilitate comparison. The position of each measurement is indicated at the top of each moment-curvature graph. For the center of the stub, the value of moment at the face of stub is used for plotting the corresponding graph. In all specimens the amount of curvature measured at the center of the stub was small.

Since the curvature measurement was made using strain gages, whenever gages went out of range the curvatures were no longer obtainable. Although precautions were taken to protect strain gages, in some cases they were damaged during casting and were not functional. The
percentage of malfunctioning gages was higher for specimens containing fibers. The presence of fibers and the additional vibration during casting may have caused the damage.

In general, higher values of curvature occurred at the face of stub where the moment was larger. The lowest curvature values were recorded at the stations farther away from the face of stub.

Specimen CC4-X was the only specimen where all of the 14 strain gages in the longitudinal bars remained functional throughout the cyclic test. The north beam, which controlled the loading protocol, experienced larger curvature than the south beam, with the highest values occurring at the face of stub. This is in agreement with the higher values of deformations (drift ratio) for the north side beam (Figure 4.18 and Figure 4.19). All of the curves presented, especially at the face of stub, indicated that large nonlinear deformations are associated with yielding of the longitudinal bars. From Figure 4.46 one can infer that the response of the section at the center of the stub was linear.

Measured moment-curvature relationships in Specimen UC4-X are shown in Figure 4.47. The values of curvature in the south beam, at stations located 4-in. or more from the face of stub, were not measured because the strain gages were not functional at these locations. Except at the center of stub, nonlinear response was apparent at other locations. High values of curvature were also measured at the intermediate station of the north beam. The higher values of curvature at the north beam are consistent with the higher drift ratios experienced by the north beam.

The curvature values were measured only at four stations in Specimen UC4-F (see Figure 4.48) and the values of curvature at the south face of stub, were not captured. Unlike the first two specimens the moment-curvature relationship for the station farther away from face of the stub (north beam) indicates a nearly linear response at this station. This observation in addition to the visual appearance of Specimen UC4-F during the test (Figure 4.8) shows that fibers were effective in transferring tensile loads and preventing the spread of damage from the face of stub.
towards the end supports. Nonlinear behavior was observed in stations close to the faces of stub and high values of curvature at these locations are consistent with visual observations during the test.

Figure 4.49 shows the moment-curvature relationships at four different locations in Specimen UC2-F. Nonlinear behavior at the south beam close to the face of stub was observed and large values of curvature both in the positive and negative directions were obtained in the south face of the stub (~ 0.005 in\(^{-1}\)). Due to saturation of the measurements at this location during the 4%-drift cycle, higher values of curvature were not recorded. However, in the north beam, measured curvatures were not as large and in the second station from the face of stub the behavior was nearly linear.

Moment-curvature relationships for Specimen CC2-F are shown in Figure 4.50. This was the first beam tested. Due to limitations of the data acquisition system during early stages of the testing program, only two stations were used. Nonlinear behavior was detected in both the south and north beams.

Moment-curvature relationships obtained for Specimen CC4-X$ (see Figure 4.51) were very similar to those of Specimen CC4-X. Except at the central stub, nonlinear behavior was observed at all stations. For the south beam, the station at the face of stub experienced the largest curvature (~ 0.006 in\(^{-1}\) in the negative direction).

The largest curvature (~ 0.008 in\(^{-1}\)) was observed in Specimen UC2-F$ (see Figure 4.52). Similar to the other HPFRC specimens, nearly linear response was detected in the stations 8 in. away from the face of the stub.
4.3.3 Measured Strains in Longitudinal Reinforcement

Longitudinal bars were instrumented with electronic strain gages as described in Appendix A. These strain gages were used to study the variation of strains and stresses along the length of the specimen. Longitudinal strains were measured for as long as the gages were functional, typically below 5% elongation. In this section, the measured strains are plotted against the beam drift ratio and shear. For all specimens, the maximum absolute negative strains (compression) were small, typically below yield. For the cyclic loads below tensile yielding of the longitudinal bars, small compression strains developed in the concrete and steel bars. After yielding of the tension reinforcement, large residual tensile strains remained in the bars even after unloading, a clear indication that large compression forces in the steel were shared with the surrounding concrete.

Figure 4.53 and Figure 4.54 show the measured strain values for Specimen CC4-X. All of the strain gages remained functional throughout the cyclic load. Figure 4.53 shows the progression of yielding in the longitudinal bars. For instance, the bottom reinforcement in the south beam first yielded close to the face of stub at approximately 1.5% drift ratio. Later, at a drift ratio of about 2%, the second strain gage located 4 in. away from the face of the stub showed yielding of the same bar and finally at 3% drift ratio, yielding occurred 8 in. away from the face of stub. Similar observations apply to the top bar and also to the top and bottom reinforcement of the north beam. Figure 4.53 suggests that upon yielding, a significant increase in tensile strain is associated with a very small increase in drift. Figure 4.54 also shows that after yielding, the applied force remains nearly constant. The small decrease in force in subsequent cycles is likely due to damage of the concrete cover causing a reduction of the internal lever arm. However, upon reaching the strain hardening region of the tensile reinforcement, the force slightly increased. At cycles beyond 3% drift ratio, all longitudinal bars had yielded, an indication that the plastic hinge
length for both beams was at least 8 in. (i.e., the effective depth of the beam, \(d\)). Figure 4.54 also indicates that at the center of the stub, both the top and bottom reinforcement responded linearly.

Figure 4.55 and Figure 4.56 show the measured strain values for Specimen UC4-X obtained from functional strain gages. Figure 4.55 demonstrates the progression of yielding in the longitudinal bars for the north beam. A trend similar to the one for Specimen CC4-X is observed for this specimen. Figure 4.55 shows that upon yielding, a significant increase in the tensile strain was associated with a very small increase in drift. However, because of the different stress-strain curve of Grade-97 bars, the amount of increase in drift values associated with strain increment (shortly after yielding) was higher than that of Specimen CC4-X (refer to Figure 4.53). Figure 4.56 also shows that after yield, the applied force remained nearly constant. During the final cycles, all strain measurements in the longitudinal bars (except at the center of the stub) indicated yielding occurred even at 8 in. from the face of stub. This observation suggests that the plastic hinge length for both beams was at least 8 in. (i.e., the effective depth of the beam, \(d\)).

Figure 4.57 and Figure 4.58 show the measured strain values for Specimen UC4-F. At the beginning of the test only a limited number of strain gages were active and therefore limited measurements were obtained. Figure 4.57 shows two distinctive characteristics of adding fibers to the concrete mix (to achieve strain hardening HPFRC). First, measurements from strain gages installed at 8 in. away from the face of stub indicate that the reinforcement had a nearly linear response at that section throughout the entire cyclic part of the test. This suggests that unlike Specimens CC4-X and UC4-X, the plastic hinge length in Specimen UC4-F was shorter than the effective depth, \(d\), (i.e., less than 8 in.). Second, the active strain gage at the face of stub in the south beam (controlling beam, top reinforcement layer) reached 5% strain in the first cycle of 5% drift ratio (note in Figure 4.55 that the maximum strain in Specimen UC4-X approached 4% during the cycle of 5% drift ratio). These measurements, consistent with the visual observations, indicate that adding fibers resulted in concentration of plastic curvature in a smaller region.
Similar to the previous specimens, upon yielding, a significant increase in tensile strain was associated with a small increase in drift. Figure 4.58 also shows that unlike Specimens CC4-X and UC4-X, the force slightly increased after yield. This is likely due to the added tensile resistance of the fiber reinforced matrix. Several cycles after the occurrence of first yield, the applied force decreased very likely due to pullout of some fibers. Figure 4.58 also indicates that both the top and bottom reinforcement within the stub were in the linear range.

Measured longitudinal strains for Specimen UC2-F, were very similar to those of Specimen UC4-F (Figure 4.59 and Figure 4.60). In the south beam, both strain gages adjacent to the face of stub at the top and bottom of the section went out of range during the test while the gages at 8 in. away from the stub barely exceeded the yield strain. The plastic hinge length in this specimen was less than 8 in. The reinforcing bars at the center of the stub showed a linear behavior similar to the other test specimens. The 50% reduction of transverse reinforcement had no apparent effect on the measured longitudinal strains.

Figure 4.61 and Figure 4.62 show the measured strain values for Specimen CC2-F. This specimen was only instrumented with four strain gages to measure strains in the longitudinal bars. All four measurements indicated nonlinear response at 4-in. (or d/2) away from the face of the stub.

Figure 4.63 and Figure 4.64 show the measured strain values for Specimen CC4-X$. The response was similar to that of Specimen CC4-X. At cycles beyond 3%, all strain gages on the longitudinal bars indicated that yielding extended 8 in. away from the face of the stub for both beams (i.e., the effective depth of the beam, d).

Measured longitudinal strains for Specimen UC2-F$, were very similar to those of Specimens UC4-F and UC2-F (see Figure 4.65 and Figure 4.66). Both strain gages adjacent to the face of stub at the top reinforcing layer went out of range during the test while the gages 8 in. away from the stub experienced much smaller strains. In this specimen the plastic hinge length
was less than 8 in. Measurement on the reinforcing bars located at the center of the stub showed a linear behavior similar to the other test specimens.

### 4.3.4 Measured Strains in Transverse Reinforcement

For each beam, the first four hoops away from the stub were instrumented with electric resistance strain gages (Figure 3.7). The second hoop in each beam was instrumented with two gages on opposite vertical legs and the rest of the instrumented hoops had only one gage on a vertical leg. The recorded hoop strains are plotted against drift ratio and applied shear in Figure 4.67 to Figure 4.78. A horizontal line indicating the yield strain of hoops (i.e., \( f_y = 2350 \text{ microstrains} \)) is included in these figures. Overall comparison of these graphs indicates that in most cases higher transverse strains were measured in the hoops closer to the face of stub. However, in a few cases, higher strains were measured in hoops 7 in. away from the face of stub. In general, all strains measured in the transverse reinforcement were below 0.4% even during the final cycles of the test. For this small range of strains, the location of an inclined crack greatly influenced gage readings. For example, the measured strains at the two legs of the second hoop of Specimen CC4-X on the south beam (Figure 4.67) indicate that one leg of the hoop remained elastic throughout the cyclic test while the other leg reached yielding in the cycles beyond 3% drift.

Measured transverse strains for Specimen CC4-X are shown in Figure 4.67 and Figure 4.68. The first two hoops on both sides of the stub yielded at large cycles after yielding of the longitudinal reinforcement. However, the yielding in transverse hoops is localized and in all hoops the strains are below 0.4%. In this specimen the hoops in the plastic hinge region experienced the highest values of strain. The occurrence of localized yield in hoops suggests that
the cracks were inclined more than the 45 degrees (with respect to the horizontal direction) assumed in the design of shear reinforcement.

Transverse strains were not measured in Specimen UC4-X due to limitation of the data acquisition system at the start of the test.

Figure 4.69 and Figure 4.70 show the measured transverse strains in Specimen UC4-F. The presence of fibers prevented the north beam from reaching hoop strains above 0.1%. The hoops on the south beam remained nearly elastic during the cyclic test. The hoops on the south beam barely yielded during the final cycle of the test.

Specimen UC2-F had only half of the required transverse steel according to Chapter 21 of ACI 318-08 (2008). The measured transverse strains in this specimen indicated yielding occurred only during the cycles of 5% drift ratio (Figure 4.71 and Figure 4.72). The north beam had a behavior similar to the north beam of Specimen UC4-F except that higher hoop strains were measured in UC2-F due to the reduction in the number of hoops.

Specimen CC2-F had only two hoops instrumented with strain gages. The gage on the north side malfunctioned from the beginning of the test. Figure 4.73 and Figure 4.74 show the values of strain measured on the first hoop of the south beam. In this hoop, the strains were below the yield strain.

Figure 4.75 and Figure 4.76 show the measured transverse strains in Specimen CC4-X$. Due to the asymmetrical layout of the longitudinal reinforcement, higher values of strain were obtained in the positive direction where the specimen was stronger. These strain values are comparable to the strains measured in Specimen CC4-X. Localized yield events were observed in the first two hoops of the south beam with values below 0.4% throughout the test.

Measured hoop strains for Specimen UC2-F$ are presented in Figure 4.77 and Figure 4.78. The measured hoop strains in the first hoop of the south beam were similar to those of Specimen UC2-F for the positive direction of loading. However, the second hoop in the north
beam experienced values of strain just below 4% in the positive direction, comparable to the values obtained in the specimens without fibers with hoops spaced at $d/4$.

**4.3.5 Measured Crack Width**

For many years concrete members were designed based on working stresses and therefore only small cracks due to flexure and shear were developed during the service life of structures. When using high strength steel reinforcement, higher stresses are induced in the tensile reinforcement even during service conditions, and if appropriate measures are not taken, wider cracks are likely to occur. Occurrence of wider cracks may not be aesthetically acceptable and may potentially increase the possibility of corrosion of the reinforcing bars. In versions of ACI 318 Code before 1999, provisions were given for distribution of longitudinal reinforcement based on a calculated maximum crack width of 0.016 in. Provisions in ACI 318-08 (2008) do not refer to a specific crack width.

Figure 4.79 to Figure 4.90 show the measured crack widths at the peak of each cycle for all tested specimens except Specimen CC2-F. Specimen CC2-F was painted with several layers of lime-based paint which made it difficult to measure crack widths accurately. In these figures a horizontal line indicates the 0.016 in. crack width limit implied in pre-1999 ACI 318 Codes. The maximum crack widths due to flexure and shear were measured at positive and negative peaks of each cycle. In the figures, the size of flexural cracks is shown by solid lines while the size of shear cracks is represented by dashed lines.

Comparing the figures for Specimen CC4-X and Specimen UC4-X reveals that crack widths were slightly larger in UC4-X for a given cycle (targeting identical drift). However, the difference is small. During cycle #10, CC4-X had crack widths equal or below 0.016 in. while UC4-X had crack widths as high as 0.02 in.
Adding fibers helped reduce the crack widths. All HPFRC specimens with Grade-97 rebar showed a crack width equal or below 0.016 in. at cycle #10 (0.6% drift ratio) similar to RC specimens with Grade-60 rebar.

A more detailed crack-width comparison between different specimens is presented in Section 4.6.8.

4.4 Shear Deformations

To calculate the shear component of the deformations, a grid of Whittemore points was used (see Figure A.35). These components were derived based on the measured angular changes in the Whittemore grid. One distorted square of the grid at an arbitrary stage of test is shown in Figure 4.91. It was assumed, as illustrated in the figure, that any angular change had three components:

1. Change in angle due to bending curvature
2. Change in angle due to shear distortion
3. Change in angle due to core expansion

The core expansion is more pronounced in concrete columns and proportional to axial load (Pujol, 2002). In beams designed with the intention that the transverse reinforcement carries the shear without yielding the amount of core expansion is assumed negligible. Measured strains in the transverse reinforcement reported in the previous sections support this assumption. Therefore, only the angular changes related to bending and shear are considered here.

The average shear distortion angle is calculated as follows:

\[ \Delta A = A_n - A_i \cong \frac{\theta}{2} + \nu \]  

(4.1)
\[ \Delta B = B_n - B_i \equiv \frac{\theta}{2} - \nu \]  
\[ \Delta A - \Delta B = 2\nu \]

Where subscript \( n \) is used for the measured angle in step \( n \) of the test and subscript \( i \) refers to the initial value.

Using the law of cosines, angles \( A \) and \( B \) are calculated using:

\[
A = \cos^{-1}\left(\frac{h_t^2 + v_t^2 - d_2^2}{2h_t v_t}\right)
\]

\[
B = \cos^{-1}\left(\frac{h_t^2 + v_r^2 - d_1^2}{2h_t v_r}\right)
\]

It was assumed that the shear deflection mainly occurred close to the face of stub.

Therefore, to calculate the contribution of shear from each square, the average calculated shear distortion for that square was multiplied by its nominal horizontal dimension (i.e., 6 in.). The total shear deflection was calculated by summing the contribution of squares 2 and 3 (Figure 4.92):

\[
\Delta_{\text{shear}} = (v_2 + v_3) \times 6 \text{ in}
\]

The portion of specimen not instrumented with the grid points was assumed not to contribute to shear deformations. The calculated shear displacement was then divided by the total shear span (i.e., 24 in.) to obtain the contribution of shear component to the total drift ratio.

The measured components of shear drift for the controlling beam of each specimen are plotted against the total drift in Figure 4.93 to Figure 4.99. The shear component of drift is predominantly available for drift ratios below 1%. In most cases the early occurrence of cracks
crossing the Whittemore gages detached the gage (typically after cycle #8). Specimens UC2-F and CC2-F allowed readings up to a drift ratio of nearly 3%.

4.5 Stiffness Reductions

To study the influence of the yield strength of reinforcement and of the presence of fibers on the stiffness of concrete beams, several stiffness measures were calculated and graphed at various stages of the test.

By definition, stiffness is the required force to cause a unit deformation. Stiffness values were calculated using the measured force-displacement curves. In this study, four different stiffness values were determined for each of the hysteretic loops:

- Peak-to-peak stiffness, \( K_p \): measured from maximum displacement to minimum displacement;
- Zero-drift stiffness, \( K_0 \): measured from zero-force to zero displacement;
- Reloading stiffness, \( K_r \): measured from zero-force to maximum displacement;
- Unloading stiffness, \( K_u \): measured from maximum displacement to zero force;

These stiffness definitions are illustrated in Figure 4.100.

4.5.1 Peak-To-Peak Stiffness

Figure 4.101 shows the stiffness reduction with increasing drift. The vertical axis shows the value of stiffness, while the horizontal axis shows the number of cycles. In general, increasing the number of cycles led to lower values of stiffness \( K_p \) even in successive cycles that reached an identical drift ratio. This stiffness measure is driven by the peak-to-peak displacement values per cycle. Tracking the value of the peak-to-peak stiffness helps identify the specimens that
experienced the largest drop in strength between cycles. For example, when comparing CC4-X$ with UC2-F$, their values of $K_p$ are nearly identical near the end of the test, but not during early cycles. The data in Figure 4.101 suggest that Specimen UC2-F$ experienced a larger strength reduction than Specimen CC4-X$. It is important to note that both of these specimens have the same $\rho'$ to $\rho$ ratio.

Since this type of stiffness uses only two points at the peak displacements of each hysteretic loop, it gives very limited information about hysteresis. In order to gain a better understanding on the shape of the hysteretic loop, other stiffness measures were investigated.

4.5.2 **Zero-Drift Stiffness and Reloading Stiffness**

Reloading generally refers to the stage of loading from a residual displacement at the start of a loading cycle to a new target displacement.

The ratio of the initial reloading stiffness, $K_0$, near zero drift, to the secant reloading stiffness, $K_r$, when compared between different cycles helps identify the cycles experiencing a “pinched” hysteresis. The lower the $K_0$-to-$K_r$ ratio, the higher the pinching effect. Figure 4.102 shows representative $K_0$-to-$K_r$ ratios for various stages of loading.

All specimens were near yielding during cycle #15 with a target drift ratio of 1.5%. Figure 4.103 shows that the stiffness ratio $K_0/K_r$ is generally less than one. The figure omits the data associated with cycles #17 to #19 of Specimen UC2-F because it deviated from the loading protocol. Figure 4.103 suggests that the pinching effect does not dramatically increase with increasing drift ratios. One could also infer that HPFRC specimens had less amount of pinching when compared with RC specimens (i.e., higher stiffness ratios correspond to HPFRC specimens). A more detailed stiffness comparison between different specimens is presented in Section 4.6.7.
4.5.3 Unloading Stiffness

The unloading stiffness, $K_u$, generally refers to the secant stiffness (measured in the shear vs. drift curve) from the maximum drift of a loading cycle to the point of zero shear force (Figure 4.100). Figure 4.104 shows the calculated $K_u$ for all specimens. In cycles before flexural yielding (typically cycles #1 to #15) the unloading stiffness is sensitive to the amount of cracking but exhibits a nearly constant value between cycles #10 and #15. For loading cycles beyond flexural yielding (cycles #15 to #25), the unloading stiffness decreased with increasing drift. The HPFRC specimens showed a slightly higher unloading stiffness than the RC specimens. A more detailed stiffness comparison between different specimens is presented in Section 4.6.7.

4.5.4 Rate of Stiffness Reduction

The unloading stiffness is generally related to the secant stiffness to the yield point (Otani, 1981). For reinforced concrete members, Takeda (1970) defined the unloading stiffness, $K_u$, using:

$$K_u = K_y \left(\frac{D_y}{D_{max}}\right)^\alpha$$

(4.7)

Where

$K_y$ is the secant stiffness to the yield point;

$D_y$ is the yield displacement;

$D_{max}$ is the maximum displacement reached in the unloading quadrant;

$\alpha$ is the unloading stiffness-reduction parameter.
The values of $\alpha$ for different specimens (where data were available) are derived in Figure 4.105 to Figure 4.109 using

$$\ln(Ku/Ky) = \alpha \ln(Dy/D_{max})$$  \hspace{1cm} (4. 8)

The data suggest that for RC or HPFRC beams reinforced with Grade-60 bars, a value of $\alpha = 0.4$ is adequate; for RC or HPFRC reinforced with Grade-97 bars, $\alpha = 0.5$ is more appropriate.

### 4.6 Response Comparison

In this section, different test beams are compared in more detail to determine the effect of various parameters on the mechanical behavior of these beams. Because Specimen CC4-X was detailed as a beam of special moment frames according to Chapter 21 of ACI 318-08 (2008), it is used as a reference to evaluate the response of the other specimens.

#### 4.6.1 Maximum Moments and Deformation Capacities

The maximum measured shear and drift ratios are presented in Table 4.1. All specimens were designed to have nearly identical flexural strength (neglecting the effect of fibers). Therefore, all specimens had nearly the same $A_s f_y$. Specimens reinforced with Grade-60 bars had $A_s f_y = 144$ kips, while those with Grade-97 had $A_s f_y = 152$ kips. Figure 4.110 shows the maximum applied shear for each specimen. The maximum strength of the control Specimen, Specimen CC4-X, corresponds to an applied shear force of 54.5 kips. Replacing the longitudinal reinforcement with high strength steel in Specimen UC4-X resulted in a maximum applied shear
of 52.5 kips. The small difference in strength for these two specimens is within expected variability. The average strength for the three RC specimens was 54.4 kips and the average value obtained for the four HPFRC specimens was 63.5 kips. Adding fibers, on average, increased the strength by 17%. The average strength for HPFRC specimens reinforced with ultrahigh strength steel (Grade-97 bars) was 61.8 kips which indicates an 18% increase when compared to the strength of 52.5 kips of UC4-X.

According to Table 4.1 all specimens had drift ratio capacities above 10%. This amount of deformation overly exceeds the required drift ratios in practical design of structures.

Failure did not occur for any specimen made of conventional Grade-60 rebar within the range of available stroke of the test actuator. In Specimen UC4-X, the ultrahigh strength bars fractured at a drift ratio of about 16%. For other specimens reinforced with ultrahigh strength steel, the addition of fibers resulted in fracture of the longitudinal bars at an average drift ratio of about 11%. This reduction is mainly due to the curvature concentration and accumulation of strains in the plastic hinge region of HPFRC specimens. As mentioned in previous sections of this chapter, the plastic hinge length was shorter in HPFRC than in RC.

### 4.6.2 Hysteresis

The shear-drift hysteretic curves of the tested specimens are shown in Figure 4.18 to Figure 4.31. In this section, the hysteretic response of each specimen is compared with each other.

Hysteretic curves of Specimens CC4-X and UC4-X are compared in Figure 4.111. In general, both specimens had similar behavior. Specimen CC4-X showed a slight strength gain after yield while Specimen UC4-X had a nearly flat post-yield shear vs. drift response. This post-yield behavior is highly dependent on the stress-strain curve of the longitudinal reinforcement.
Specimen UC4-X was capable of producing stable hysteretic loops but slightly narrower than those of Specimen CC4-X.

The addition of high-performance fibers helped in widening the hysteretic loops. The behavior of Specimen CC4-X and UC4-F are compared in Figure 4.112. Specimen UC4-F was similar to Specimen UC4-X but with high-performance fibers. In addition to widening of the hysteretic loops, fibers were effective in increasing the peak strength of the beam up to a drift of about 3% but mainly in one direction of loading (Figure 4.113). However, after experiencing the overall peak strength near 3% drift, the strength was reduced to a value approaching that of Specimen UC4-X (without fibers). This unstable increase in strength was also observed in the other two HPFRC specimens reinforced with the Grade-97 bars but not so if reinforced with Grade-60 bars (Specimen CC2-F) where strain-hardening of the rebar makes up for fiber pullout.

According to Figure 4.114 the reduction of transverse reinforcement by 50% in Specimen UC2-F, made no significant change in the behavior of this specimen compared to Specimen UC4-F. The HPFRC specimens were effective in providing shear capacity and confinement. Figure 4.115 compares the hysteretic behavior of Specimens CC2-F and CC4-X. Again, fibers were effective in transferring stresses and the 50% reduction in transverse reinforcement did not impair Specimen CC2-F. Adding fibers increased the strength of CC2-F in relation to CC4-X. Note that, unlike the specimens with Grade-97 bars, the increased strength was retained even at large drift ratios. The difference is mainly due to the strain-hardening characteristics of the Grade-60 reinforcing bars.

Hysteretic loops of Specimens CC4-X and CC4-X$ are compared in Figure 4.116. A reduction of 50% in the top longitudinal bars directly translated in a reduction of strength in the negative direction (pulling of the actuator). However the deformation capacity of the specimen was not affected by this change. In other words, for the tested RC specimens the deformation capacity of the beams was not sensitive to the compressive to tensile steel ratio ($\rho'/\rho$ of 0.5 or 1).
Figure 4.117 shows the hysteretic loops of Specimens CC4-X$ and UC2-F$. These two specimens had $\rho'/\rho$ of 0.5. The addition of high-performance fibers in Specimen UC2-F$ helped attain a higher strength in both directions. However, at drift ratios approaching 5%, the strength of both specimens was nearly the same. Fibers were effective in compensating for the reduction of the transverse reinforcement. A close comparison between the hysteretic loops of these two specimens reveals that Specimen UC2-F$ had a slightly larger area enclosed by the hysteretic loops.

The envelopes of the shear-drift curves (all-positive quadrant) for the controlling beams of each specimen are shown in Figure 4.118. At large drift ratios, all specimens, except for CC2-F, have similar envelopes. The HPFRC specimens showed an increase in strength shortly after flexural yielding, however, for drift ratios above 5% their strength reduced to a value comparable to the strength of RC specimens. Specimen CC2-F showed a pronounced strain hardening behavior after yielding, very likely due to the longitudinal bars responding in the strain hardening region of their stress-strain characteristics.

4.6.3 Moment-Curvature Relationships

Measured moment curvature graphs for all specimens are shown in Figure 4.46 to Figure 4.52. Considering the sensitivity of local curvature to the existence of major cracks at the location of the strain gages, it may be stated that all RC specimens had very similar behavior. Thus, using ultrahigh strength steel only slightly affected the curvature gradient along the length of the beam. However, the addition of fibers had a more pronounced effect on the gradient of curvature. Compared with RC specimens, the HPFRC specimens showed relatively higher curvature values in the vicinity of the stub and much smaller curvature values 8 in. away from the face of the stub.
This fact may be attributed to the occurrence of a dominant flexural crack in the plastic hinge region of the HPFRC specimens.

### 4.6.4 Strain in Longitudinal Reinforcement

Measured longitudinal strains for the tested specimens are presented in Figure 4.53 to Figure 4.66. Based on the gradient of the measured longitudinal strains along the length of the beams, the plastic hinge length was not less than 8 in. for both Grade-60 and Grade-97 bars.

The post yield behavior of the specimens was affected by the stress-strain curve of the longitudinal bars. Figure 4.119 and Figure 4.120 compare the measured strain at the face of stub for Specimens CC4-X and UC4-X. In general, Specimen UC4-X reinforced with ultrahigh strength steel (Grade 97) had higher strain than CC4-X for the same drift ratio (Figure 4.119). The measured strains for Specimen CC4-X had a sudden increase after yielding (represented by a nearly vertical line in Figure 4.119). After a few post-yield cycles, strain in the longitudinal bars for Specimen CC4-X became proportional to the drift ratio. After yielding, the slope of the strain vs. drift ratio relationship for Specimen UC4-X was nearly constant.

The strain vs. shear curve of Specimen CC4-X (Figure 4.120) shows a nearly constant shear followed by strain hardening, while Specimen UC4-X showed a small increase in shear shortly after yield.

The addition of high-performance fibers dramatically affected the measured strains in the longitudinal bars. At the face of stub, where the highest moment occurred, higher values of strain were obtained for HPFRC specimens (Figure 4.121). A close inspection of this figure reveals that the difference between maximum and minimum measured strains per cycle is greater in the HPFRC specimens. Figure 4.122 shows that the slope of the strain vs. drift curve is greater in the HPFRC specimens at 8 in. away from the face of the stub (compared to the measurements at the
face of stub) with smaller values of strains in the HPFRC specimens compared to the RC specimens.

### 4.6.5 Strains in Transverse Reinforcement

Measured strains in transverse reinforcement are shown in Figure 4.67 to Figure 4.78. Since the hoop strains were not measured in Specimen UC4-X, no direct comparison can be made on the effect that the use of ultrahigh strength longitudinal reinforcement has on hoop strains. However, having observed similar overall behavior for Specimens CC4-X and UC4-X, very likely the above mentioned effect is minimal.

In general, all measured hoop strains were small but yielding of the hoops was detected in some specimens. Adding fibers generally decreased the maximum measured hoop strain. The HPFRC specimens, CC2-F, UC2-F, and UC2-F$ with 50% of the required transverse reinforcement, showed measured strains slightly higher than Specimen UC4-F with 100% of the required shear reinforcement (following Chapter 21 of ACI 318-08). However, the measured strains, in HPFRC specimens (with hoops at $d/2$) did not exceed the values obtained for RC specimens (with hoops at $d/4$). Therefore fibers may safely reduce the amount of required transverse reinforcement.

### 4.6.6 Shear Deformations

Calculated values of the shear component of drift are presented in Figure 4.93 to Figure 4.99. These calculated values are based on measurements made (on the controlling beam of each specimen) for drift values predominantly below 1% with a couple of cases for drift values approaching 3%. Figure 4.123 shows the average contribution of shear to the total drift for each
specimen. The average is calculated for the readings reported in Figure 4.93 to Figure 4.99. The calculated standard deviation for each specimen is also noted in the same figure. The contribution of shear deformations to the total measured drift was approximately 16%.

4.6.7 Stiffness Reductions

To investigate the effect of different parameters on stiffness, results for various stiffness definitions are presented in Figure 4.101 to Figure 4.104. A comparison between different specimens is made in this section. To avoid the scatter involved during small cycles, the comparison is made for cycles starting at 1.5% drift (cycle #15).

Figure 4.124 shows the peak-to-peak stiffness values for Specimens CC4-X and UC4-X. The data suggest that the type of longitudinal bar did not affect this parameter. Both specimens had nearly identical $K_p$ stiffness values after flexural yielding and up to the end of test. Adding fibers did not severely affect this parameter (Figure 4.125). Because all specimens had the same dimensions and were proportioned to have similar flexural strength, this parameter was nearly insensitive to the type of longitudinal bar or to the presence of fibers.

As mentioned in Section 4.5.2, the stiffness ratio of $K_0$ to $K_r$ is indicative of crack openings and pinching. This ratio is compared for Specimens CC4-X and UC4-X in Figure 4.126. Overall, Specimen UC4-X reinforced with ultrahigh strength steel, showed a slightly lower $K_{p}$-to-$K_r$ ratio. Because a lower amount of steel was used in the specimen and the longitudinal bars generally experienced higher strain values (Figure 4.119), cracks in UC4-X were wider. Therefore, more deformation when reloading was needed to close the cracks. However, the lower stiffness ratio did not severely affect the overall behavior of the specimen.
Adding fibers resulted in reduced spalling and less pinching towards the end of the test as shown in Figure 4.127. A similar trend is observed when comparing Specimen CC4-X$ (without fibers) with Specimen UC2-F$ (with fibers) as shown in Figure 4.128.

Using ultrahigh strength steel resulted in slightly lower unloading stiffness values (Figure 4.129). The average amount of reduction in the unloading stiffness of UC4-X compared to CC4-X was less than 15%. This reduction in unloading stiffness was recovered by adding fibers as shown in Figure 4.130 where Specimen UC4-F shows nearly identical unloading stiffness after cycle #16.

4.6.8 Crack Widths

Measured crack widths at different stages of each test are presented in Figure 4.79 to Figure 4.90. Crack control is primarily an aesthetic condition and although ACI 318-08 (2008) does not limit the maximum width, a calculated value of 0.02 in. under service load is generally acceptable. To investigate the effect of different parameters on crack widths, the service load demand is estimated using 60% of the nominal strength of the control Specimen CC4-X. Taking the nominal flexural strength CC4-X as 110 k-ft (Table 5.3) and for a shear span of 2 ft, the shear force associated with the estimated service load is 33 kips.

To have a fair comparison between the crack widths of different specimens, the measured crack widths are graphed in Figure 4.131 and Figure 4.132 for an applied shear between 30 and 35 kips. The crack widths in these figures correspond to the first time when each beam experienced a shear force between 30 and 35 kips.

Figure 4.131 only includes the RC specimens. Using ultrahigh strength steel (Specimen UC4-X) generally resulted in increased flexural and shear crack widths at service load level.
Adding fibers dramatically reduced the width of flexural cracks under service loads (compare Figure 4.132 with Figure 4.131). In fact, fibers were effective in reducing the crack widths to values smaller than the crack widths of Specimen CC4-X. They were also very effective in reducing the shear cracks under service loads (inclined cracks in Specimen UC2-F were not visible at service load levels).
CHAPTER 5

NUMERICAL MODELING OF SHEAR-DRIFT RESPONSE

5.1 Introduction

In this chapter a procedure based on the principles of mechanics of materials is used to calculate the shear vs. drift response of the test specimens. Nonlinear moment-curvature analysis of the cross section forms the basis of the procedure.

Instead of tracking the entire hysteretic behavior of each specimen, the envelope of the shear vs. drift curve is calculated. The assumed moment-curvature relationship is based on the hypothesis that plane sections remain plane after bending (linear strain distribution) even after inelastic response. Because several factors affecting the overall behavior of the beam, such as loading history and shear-flexure interaction are neglected in this methodology, results from these analyses should be carefully interpreted.

5.2 Moment-Curvature Relationship

A computer program was developed to calculate the moment-curvature relationship of rectangular doubly-reinforced beam sections. The program assumes a value of strain in the extreme compression fiber of the section and varies the depth of the neutral axis to satisfy equilibrium. The calculated internal forces acting on the section were determined using the constitutive material models and strain compatibility. Thereafter, the internal moment acting on the section and the corresponding value of the curvature was computed. The procedure was
repeated for increments of compressive concrete strain until the maximum (user-defined) strain is reached.

Several constitutive material models for concrete and reinforcing steel were used in the analyses of different beams. In performing the analysis, the section was divided into vertical and horizontal layers. In RC specimens two different constitutive models for compression of concrete were used, one for unconfined concrete (outside of the transverse hoops) representing the shell, and one for confined concrete enclosed by rectangular hoops. In Figure 5.1, the ascending branch is adopted from the model suggested by Hognestad (1951) and the descending branch is based on the simple model suggested by Roy and Sozen (1964). A compressive strength of 6000 psi was assumed for all specimens and the tensile strength of concrete was neglected for the RC specimens but not for the HPFRC specimens.

For unconfined concrete in compression, the model by Hognestad (1951) is defined using:

For $0 < \varepsilon < \varepsilon_0$

$$f_c = f'_c \left( \frac{2\varepsilon}{\varepsilon_0} - \left( \frac{\varepsilon}{\varepsilon_0} \right)^2 \right)$$

(5.1)

Where

$$\varepsilon_0 = \frac{1.8f'_c}{E_c}$$

(5.2)

$E_c$ is the modulus of elasticity for concrete as defined in ACI 318-08 (2008):

$$E_c = 57000\sqrt{f'_c} \text{ (psi)}$$

(5.3)
For $\epsilon_0 < \epsilon < \epsilon_{c,\text{max}}$

$$f_c = f'_c (1 - z(\epsilon - \epsilon_0)) \quad (5.4)$$

The parameter $z$ controls the slope of the descending branch, a value of 110 was used for unconfined concrete.

For confined concrete, a similar model to that of unconfined concrete was used for the ascending branch. For the descending branch, a value of $z = 20$ was adopted as suggested by Roy and Sozen (1964). Figure 5.1 shows the concrete models used and Table 5.1 indicates the parameter values defining the models.

For HPFRC, it was assumed that fibers provided effective confinement to concrete in compression. Therefore, the same model described for confined concrete (Figure 5.1) was used for HPFRC in compression. In other words, a $z$ value of 20 was used for both the shell and concrete core (Table 5.1).

HPFRC is effective in transferring tensile stresses. The simplified model shown in Figure 5.2 was used to define the stress-strain relationship of HPFRC in tension. This model was adapted from the multilinear model suggested by Chompreda (2005). The model is defined using

For $\epsilon \leq \epsilon_r$

$$f_c = E_c \epsilon \quad (5.5)$$

Where $\epsilon_r$ is defined by

$$\epsilon_r = \frac{f_r}{E_c} = \frac{7.5 \sqrt{f'_c}}{E_c} \quad (5.6)$$

For $\epsilon_r < \epsilon < \epsilon_m$
\[ f_c = f_r + \frac{(af_r)}{\varepsilon_m - \varepsilon_r} (\varepsilon - \varepsilon_r) \]  
(5.7)

Where

\[ \varepsilon_m = b\varepsilon_r \]  
(5.8)

For \( \varepsilon_m \leq \varepsilon \leq \varepsilon_p \)

\[ f_c = f_m \]  
(5.9)

Where

\[ \varepsilon_p = c\varepsilon_m \]  
(5.10)

For \( \varepsilon_p < \varepsilon \)

\[ f_c = f_m - \frac{f_m - .01f_c'}{.015 - \varepsilon_p} (\varepsilon - \varepsilon_p) \geq 0 \]  
(5.11)

Where \( a, b, \) and \( c \) were set to 0.25, 10, and 2, respectively. The relative magnitudes of these parameters define the breakpoints of the multilinear stress-strain model. The adopted values were derived from direct tension test data, presented by Liao et al. (2006), for HPFRC dog-bone specimens prepared using the same concrete mix proportions and fibers as in the present study.

The conventional Grade-60 steel was modeled using three different segments as shown in Figure 5.3. Young’s modulus, \( E_s \), was defined equal to 29000 ksi. The yield stress of rebar was defined as 65 ksi and the tensile strength was taken as 98 ksi. These values are based on tests of sample reinforcing bars (Table A.3). The model for Grade-60 steel is defined using:

For \( \varepsilon \leq \varepsilon_y \)
For $\epsilon_y < \epsilon \leq \epsilon_{sh}$

$$f_s = f_y$$

For $\epsilon_{sh} < \epsilon \leq \epsilon_{su}$

$$f_s = f_y + (f_u - f_y) \left(2 \left( \frac{\epsilon - \epsilon_{sh}}{\epsilon_{su} - \epsilon_{sh}} \right) - \left( \frac{\epsilon - \epsilon_{sh}}{\epsilon_{su} - \epsilon_{sh}} \right)^2 \right)$$

The strain corresponding to strain hardening, $\epsilon_{sh}$, was assumed to be 0.015 and the ultimate strain, $\epsilon_{su}$, was taken as 0.1.

Specimens #1 and #6 were reinforced with Grade-60 rebar. To calculate the moment vs. curvature for these two specimens the aforementioned concrete and steel models were used. Figure 5.4 and Figure 5.5 compare the calculated moment vs. curvature relationships with the measured response of Specimen CC4-X (presented in Chapter 4). Good agreement between the calculated and experimental results was obtained. Similar results were also obtained for Specimen CC4-X$ as shown in Figure 5.6 and Figure 5.7.

The ultrahigh strength steel (Grade 97) was modeled using a bilinear curve (Figure 5.8). In the model, a yield strength of 97 ksi and a tensile strength of 117 ksi were used. The Young’s modulus for this type of steel was taken as 29000 ksi.

The aforementioned concrete models and the bilinear model of ultrahigh strength steel were used to calculate the moment vs. curvature for Specimen UC4-X. The results are compared with the measured values in Figure 5.9 and Figure 5.10. A good agreement between the numerical and experimental results was obtained. The numerical model slightly underestimated the maximum moment in positive and negative directions. An effective depth of 8 in. was assumed
for all specimens. The slight underestimation of ultimate strength may be explained by small differences in the location of the longitudinal reinforcement and the coherent randomness involved in material properties.

Using the aforementioned models for HPFRC and the bilinear model for ultrahigh strength steel, the moment curvature of the HPFRC beams reinforced with Grade-97 bars was calculated. The results are compared with the measured values in Figure 5.11 to Figure 5.15. In all cases the calculated maximum positive flexural strength reasonably estimated the measured strength. The maximum negative moment tended to be overestimated. The measured data indicate that once the HPFRC beam experienced its maximum strength in one direction it was not capable to gain the same strength in the opposite direction. In all cases the calculated values were similar to the measured values.

The model shown in Figure 5.3, for conventional steel, was used in combination with the HPFRC model to simulate the moment-curvature response of Specimen CC2-F. Unfortunately, due to the limited instrumentation of Specimen CC2-F, a direct comparison between the measured and calculated data was not possible. Figure 5.16 shows the calculated moment-curvature relationship for this specimen. The increase in the strength of specimen at high curvature values is in agreement with the fact that Specimen CC2-F continued to gain strength in the final push (Figure 4.36) and that the maximum strength of this specimen was obtained at the end of the test.

Calculated moment-curvature relationships for Specimens CC4-X, UC4-X, UC4-F, and CC2-F are compared in Figure 5.17. These four specimens invoke the different mathematical models that represent the stress-strain relationships of the materials used in the experiments. Because of the reasonable agreement between numerical and experimental results, Figure 5.17 provides a good basis for comparison of the behavior of different specimens regardless of coherent random scatter in experimental data. From this figure one can infer that the post-yield
behavior of the tested beams is highly influenced by the post-yield characteristics of the reinforcing steel. The RC beams reinforced with Grade-97 steel (Specimen UC4-X) exhibit a nearly flat post-yield response, while those reinforced with conventional reinforcement (Specimens CC4-X and CC2-F) exhibit a strain hardening stage in their response. Adding 1.5% volume fraction of steel-hooked fibers (Specimens UC4-F and CC2-F), increases the strength of the beams. The additional strength is primarily due to the tensile stress contribution of the HPFRC and it is nearly lost soon after reaching the peak strength; however, the beam reinforced with Grade-60 steel (Specimen CC2-F) regains strength at large curvatures.

The calculated and measured yield moments of the beams are reported in Table 5.2. The measured yield moment was defined using the shear vs. drift ratio curves presented in Chapter 4. The yield point was located where a significant change (of 80% or more) in the slope of the shear vs. drift curve occurred. In general, experimental results showed a smaller yield moment than the calculated one. The average ratio of measured yield moment to calculated yield moment was 0.97 with a standard deviation of 0.06. Higher ultimate moment values were generally obtained from experimental results compared to calculated values (Table 5.3). The average ratio of measured maximum moment to calculated maximum moment was 1.07 with a standard deviation of 0.07.

Once the moment-curvature relationships were obtained, they were used to calculate rotation and displacement of the beams. Rotation was obtained by integrating the curvature along the length of the beam and displacement (drift) was obtained by double integration of curvature along the length of the member. Special definitions are required for the region where yielding occurs, particularly regarding the plastic hinge length. This procedure is defined in the following section.
5.3 Calculation of Drift

The total deflection of a concrete beam has different components. The main components considered here were flexure, shear, and slip. Basic principles of mechanics were used to estimate the contribution of each component to the total drift (defined in Appendix A). The calculated components were added together to estimate the total deflection. Drift is herein defined as the lateral (transverse) deflection of the beam between the points of zero and maximum moments. Drift ratio, is defined as the drift divided by the shear span.

5.3.1 Shear Component of Drift

The linear theory of elasticity was used to calculate the shear deflections. According to beam theory, the shear deformation of a cantilever with a rectangular section, subjected to constant shear can be determined using:

\[
\delta_v = \frac{6LV}{5A_eG}
\]

(5.15)

Where \(L\) is the span length, \(V\) is the applied shear, \(A_e\) is the effective shear area, and \(G\) is the shear modulus defined as:

\[
G = \frac{E}{2(1 + \nu)}
\]

(5.16)

Where \(E\) is the Young’s modulus of concrete and \(\nu\) the Poisson’s ratio, taken as 0.2 for concrete.

The corresponding shear component of the drift ratio is determined using:
\[
\theta_v = \frac{\delta_v}{L} = \frac{6V}{5A_e G}
\]

(5.17)

In an undamaged section, the effective shear area, \(A_e\), is the gross cross-sectional area. However in concrete beams under cyclic load, the existence of cracks and cover spalling reduces the effective shear area. Therefore, the effective shear area was taken as the area defined by the outside perimeter of the hoops (i.e., 12.25” × 7.5” = 92.5 in²).

Knowing the applied shear force, the drift ratio related to shear was calculated according to Eq. (5.17). In general, the calculated amount of shear drift was several times smaller than the measured value derived from Whittemore readings (Figure 4.93 to Figure 4.99); therefore, an empirical method based on the Whittemore measurements was used to calculate the shear component of drift.

The data in Figure 4.123 show that for the tested beams (controlling the loading protocol) the contribution of shear deformations to total drift varies between 14% and 17%. The reported averages predominantly correspond to Whittemore readings made during drift ratios between 0.2% and 1%. Based on the above data, the numerical model used in this study simply adopted a constant shear component of drift equal to 16% of the total drift.

It is important to mention that Matamoros (1999) and Chompreda (2005) have shown that the percent contribution of shear deformations to total drift increases with increased drift. This stiffness-reduction effect is not accounted for in this study due to insufficient Whittemore measurements at large drift ratios.
5.3.2 Drift Related to the Slip of Longitudinal Reinforcement

At the face of the central stub, due to strains in the longitudinal bars inside the stub, the longitudinal reinforcement undergoes elongations due to bond slip. This elongation, illustrated in Figure 5.18, contributes to the total deflection of the beam. A linear strain gradient was assumed for the longitudinal reinforcement inside the central stub, as shown in Figure 5.19. To calculate the contribution of bond slip to the total drift it is important to have a good estimate of the strain in the longitudinal reinforcement inside the stub. Figure 5.20 to Figure 5.22 show the relationship between the measured strain at the face of the stub and at the center of the stub for Specimens CC4-X, UC4-X, and UC2-F. An idealized bilinear relationship, included in Figure 5.20 to Figure 5.22, was used for numerical calculations. The slope of the initial segment (pre-yield) for Specimen CC4-X and UC4-X was taken as 90%; for Specimen UC2-F, a value of 80% was more appropriate. The data in Figure 5.20 to Figure 5.22 suggest that for Specimen CC4-X the slope of the second line segment should be 0%, while for Specimens UC4-X and UC2-F, the slopes of the second line segments should be 3% and 1%, respectively.

The bond-slip model assumed for Specimen CC4-X was also used for Specimen CC4-X$. The bond-slip model assumed for Specimen UC2-F was also used for the other HPFRC Specimens (UC4-F, CC2-F, and UC2-F$) where the breakpoint of the assumed bilinear relationship (Figure 5.22) is defined by the yield strain of the longitudinal reinforcement.

At each point of the moment-curvature diagram, the strain in the tensile reinforcement was calculated. Knowing the strain in the bar, the amount of slip (or elongation, $e$) was determined based on the assumed relationship between strains at the center of stub and at the face of stub with a linear gradient in between. The slip was calculated using the average strain inside the stub times 8-in. (half the stub length). The drift ratio due to slip was obtained by dividing the amount of slip by the distance between the two layer of reinforcement (Pujol, 2002):
\[ \theta_{\text{slip}} = \frac{e}{d - d'} \]  \hspace{1cm} (5.18)

where \( e \) is the elongation of the bar due to bond slip and \((d-d')\) is the distance between the two layers of reinforcement.

### 5.3.3 Flexural Component of Drift

If the curvature along the span of the beam is known, the rotation along the beam can be determined by integrating the curvature diagram. Similarly, the deflection can be determined by integrating the rotation. In order to find the value of curvature along the length of the beam, the bending moment diagram is derived from equations of equilibrium and the corresponding value of curvature at each point along the length of the beam was taken from the calculated moment-curvature relationships.

The curvature distribution in the plastic hinge region was assumed constant as shown in Figure 5.23. The plastic hinge length, \( L_p \), for the RC specimens was assumed equal to the effective depth, \( d \), while for the HPFRC specimens, the plastic hinge length was assumed to be half of the effective depth.

### 5.3.4 Calculated Shear-Drift Response

Adding the different components of drift (due to shear, slip, and flexure) resulted in the total calculated drift. The calculated values provide a relationship between shear and drift representing the response envelope of each specimen. When a beam is subjected to large deflection cycles, at each cycle the unrecovered longitudinal strains in the reinforcement
accumulate. The stiffness of the beam also reduces due to shear cracks and cover spalling. The loading history highly influences the shear-drift response of the specimen (Pujol, 2002). Because these effects are not explicitly accounted for, the calculated results based on the proposed model should be interpreted with care.

The simulated results for different specimens are compared to the measured shear-drift envelope (all-positive quadrant) of each specimen (Figure 5.24 to Figure 5.30). A reasonable agreement between the calculated and experimental results is observed. Therefore, the proposed models may be used to study the relative effects of different parameter values on the mechanical behavior of concrete beams.

The calculated data presented in Figure 5.24 to Figure 5.30 correspond to assumed compression strains (in the extreme fiber) as high as 3%. It is important to note that at these strain levels, the shell concrete is still partially effective in the HPFRC specimens but not so in the RC specimens.

Table 5.4 compares the calculated and measured drift ratios at yielding of longitudinal reinforcement for the tested specimens. In most cases, especially in the beams (north or south) controlling the loading protocol, there is a good agreement between the calculated and measured data.

### 5.4 Simplified Method for Calculation of Strength Using ACI 318 Code

The yield moment and the maximum moment were also estimated using the strain compatibility analysis recommended in ACI 318-08 (2008) but without limitations on the value of $f_y$. The calculated yield moment was based on the provisions in Chapter 10 of ACI 318-08 for determining $M_y$, and the calculated maximum moment was based on the provisions in Chapter 21 of ACI 318-08 for determining $M_{pr}$. Table 5.5 and Table 5.6 compare the calculated and
measured values. A maximum compression strain of 0.003 was assumed for all cases. Based on ACI 318 Code recommendations, a linear strain distribution along the height of the cross section was assumed. The equivalent rectangular stress block for concrete in compression was used ($\beta_1 = 0.75$ for $f'_c$ of 6000 psi). The yield strength of conventional bars was taken as 60 ksi and for ultrahigh strength bars a value of 97 ksi was used. The calculation of $M_{pr}$ was based on $1.25 f_y$.

According to Table 5.5 and Table 5.6, the calculated moments using the simplified method were within 10% of the measured values. These results suggest that the flexural strength formulations in ACI 318-08 (2008) may be used to estimate the strength of reinforced concrete beams reinforced with Grade-97 bars.

5.5 Nonlinear Seismic Response of Single-Degree-of-Freedom Systems

The experimental results presented in Chapter 4 show that the secant stiffness to the yield point of the measured shear vs. drift ratio is about 25% greater in specimens reinforced with Grade-60 bars than in specimens reinforced with Grade-97 bars, see Figure 4.18 to Figure 4.21. Similarly, the unloading stiffness is greater in specimens with Grade-60 bars than in specimens with Grade-97 bars (refer to Section 4.5.3).

This section investigates how changes in the hysteretic response of reinforced concrete members, due to the use of Grade-60 reinforcement versus Grade 97, affect the displacement demand in single-degree-of-freedom (SDOF) systems subjected to strong ground motions. Due to their special hysteretic response characteristics, HPFRC members are not included in this section.
5.5.1 Properties of SDOF Systems

A total of 12 SDOF systems were selected, 6 of which represent concrete beams reinforced with Grade-60 steel bars and the other six represent concrete beams reinforced with Grade-97 bars. The Grade-60 systems include 3 different periods of vibration (\( T = 0.6, 0.9, \) and 1.2 s) and 2 strength ratios (\( SR = 1/3 \) and 1/6). The strength ratio measures the nonlinear-response capacity in relation to the linear-response demand. The target design spectrum, defining the linear-response demand, was based on \( S_{DS} = 1.0 \) and \( S_{D1} = 0.75 \) as defined in ASCE/SEI 7-10 (2010). The properties of the SDOF systems considered are described in Table 5.7. Note that the Grade-97 systems were patterned after the Grade-60 systems using equivalent strength but with reduced stiffness as indicated by the difference in period of vibrations.

The force-displacement relations that characterize the SDOF systems are based on a simplified version of the Takeda hysteresis model (Takeda, 1970), as shown in Figure 5.31. The model is defined by four parameters: the initial stiffness, \( K_y \); the yield strength, \( V_y \); the post-yield stiffness, \( K_{py} \); and the unloading stiffness-reduction parameter, \( \alpha \). The modified Takeda model adopted here uses a bilinear primary curve. This model can produce displacement waveforms very similar to that of more elaborate models (Otani, 1981). The model behaves linearly elastic with stiffness \( K_y \) until the force exceeds the yield force \( V_y \). The post-yield stiffness is here defined as 5% of the initial stiffness. The stiffness during unloading from a point of maximum displacement and during subsequent reloading is equal to \( K_u \), defined using

\[
K_u = K_y \left( \frac{D_y}{D_{max}} \right)^\alpha
\]

(5.19)

where

\( K_y \) is the initial stiffness;
$D_y$ is the yield displacement;

$D_{\text{max}}$ is the maximum displacement reached in the unloading quadrant;

$\alpha$ is the unloading stiffness-reduction parameter.

A value of $\alpha = 0.4$ is assigned to the Grade-60 systems and $\alpha = 0.5$ is used to represent the Grade-97 systems. These values are in agreement with the data presented in Section 4.5. Figure 5.32 and Figure 5.33 help visualize the goodness-of-fit between the hysteresis model adopted here and the measured response of Specimen CC4-X and Specimen UC4-X. The calculated data are shown only for the nominal target displacements of 1.5%, 2%, 3%, 4%, and 5% as defined by the loading protocol (Figure 3.4). Note the reasonable accuracy obtained at the intersection with the x-axis (zero-shear), an indication that the selected values of $\alpha$ are adequate.

The viscous damping assigned to the SDOF systems is based on a damping coefficient of 2%, assumed constant (mass-proportional damping) during the calculated nonlinear response.

### 5.5.2 Ground Motions and Scaling

The SDOF systems were subjected to a suite of 10 ground motion records. The selected acceleration records, described in Table 5.8, are representative of major earthquakes in the United States. The acceleration data for each of the earthquake records are shown in Figure 5.34.

The earthquake records were obtained from the Center for Engineering Strong Motion Data (CESMD, 2010). The CESMD is part of the California Strong Motion Instrumentation Program (CSMIP) and may be found at www.strongmotioncenter.org. Although the recorded raw data are made available by the CESMD, the data used here correspond to the processed (corrected) ground acceleration.

Each of the earthquake records was scaled to a peak ground velocity of 20 in./s. The linear-response acceleration spectra for the 10 records after scaling are presented in Figure 5.35.
The figure shows that the average of the spectral accelerations, in the period range between 0.6 and 1.2 s, is within 10% of the idealized target spectrum.

5.5.3 Nonlinear Dynamic Analyses

The nonlinear seismic response is computed using the \( \beta \)-Method by Newmark (1959), a numerical time-step procedure to evaluate dynamic response. The version of the method adopted here is also known as the linear acceleration method, with \( \beta = 1/6 \). The time step used in the analyses was taken as the smaller of the accelerogram interval and 1/20 of the initial period of the SDOF system. In time intervals where a change in slope of the assumed force-displacement relationship was detected, the time steps were further reduced to improve accuracy.

5.5.4 Displacement Response Comparison

Representative force vs. displacement output is shown in Figure 5.36 for the Grade-60 systems and Figure 5.37 for the Grade-97 systems. The figures suggest that generally the calculated maximum displacement will be slightly larger for the Grade-97 systems due to their lower initial and unloading stiffnesses. Figure 5.38 compares the calculated maximum displacement for all 12 SDOF systems subjected to the suite of 10 earthquake records. The mean for the ratios of the maximum displacement of the Grade-97 systems to the maximum displacement of the Grade-60 systems was 1.09 with a standard deviation of 1.14.
CHAPTER 6
SUMMARY AND CONCLUSIONS

6.1 Summary

This investigation was aimed at studying the cyclic behavior of concrete members reinforced longitudinally with ultrahigh strength steels (yield strengths greater than 80 ksi). The main focus was on laboratory experiments for evaluating the deformation capacity and hysteretic response of beams subjected to large displacement reversals.

Chapter 2 presented a summary of previous research on the use of ultrahigh strength steel as concrete reinforcement. The summary identified relevant research on seismic applications of reinforced concrete (RC) and included recent developments on the use of high performance fiber reinforced concrete (HPFRC). In this study, HPFRC was defined as a class of fiber reinforced concrete with tensile strain-hardening behavior after first cracking and with multiple cracks at relatively high strain levels.

Chapter 3 outlined the experimental program involving seven beam specimens. Each specimen consisted of two beams spanning from a common central joint. The main experimental variables were: the type of longitudinal reinforcement (conventional Grade 60 or ultrahigh strength Grade 97), the volume fraction of steel-hooked fibers used in the concrete mix \( \nu_f = 0\% \) for RC or \( \nu_f = 1.5\% \) for HPFRC), the ratio of compressive to tensile reinforcement \( (\rho' / \rho = 0.5 \text{ or } 1) \), and the spacing of the transverse reinforcement \( (s = d/4 \text{ or } d/2) \).

The HPFRC specimens had 1.5% volume fraction of steel-hooked fibers (Dramix RC-80/30-BP by Bekaert) added to the same concrete mix \( (f_c' = 6000 \text{ psi}) \) that was used to cast the non-fiber RC specimens. All specimens were designed to have nearly identical flexural strength
and induce a maximum applied shear stress ($V/\beta d$) of about $6\sqrt{f_c'}$ (psi). The specimens were proportioned so that flexure dominated the nonlinear response under cyclic loading.

The central joint of each specimen was attached to a hydraulic cylinder mounted on a self-reacting loading frame (Figure 3.3). The loading protocol, patterned after the recommendations of FEMA 461, included two cycles for each successive displacement increment. The target drift ratios started with 0.15% and ended with 5% after 24 cycles (Figure 3.4). The specimens were subjected to a final monotonic push until failure.

The range of parameter values defining the experimental program follows:

**Beam dimensions**
- Width, $b$: 16 in.
- Depth, $h$: 10 in.
- Effective depth, $d$: 8 in.
- Shear span: 24 in.

**Concrete matrix**
- Specified compressive strength, $f_c'$: 6000 psi
- Proportions by weight: 1.0 : 2.4 : 1.9 (cement : fine : coarse)
- Maximum aggregate size: 0.5 in.
- Fiber volume fraction, $v_f$: 0 or 1.5% of Dramix RC-80/30-BP by Bekaert

**Longitudinal reinforcement**
- Specified yield strength, $f_y$: 60 ksi (Grade 60) or 97 ksi (Grade 97)
- Nominal bar diameter: #7 (Grade 60) or #6 (Grade 97)
- Bar layout: Symmetrical (8 bars) or asymmetrical (6 bars)
- Tension reinforcement ratio, $\rho$: 1.9% (Grade 60) or 1.2% (Grade 97)

**Transverse reinforcement**
- Specified yield strength, $f_{yd}$: 60 ksi
Type: #3 rectangular hoops
Spacing: \( d/2 \) or \( d/4 \)
Reinforcement ratio, \( \rho_t \): 0.34% or 0.69%

Chapter 4 compiled the measured experimental data. All specimens completed the 24 displacement cycles and were capable of retaining more than 80% of their peak strength throughout the test, including the final push with drift ratios in excess of 10%. The measured data included strain in the longitudinal and transverse reinforcement, applied force and displacement, width of cracks, and other derived quantities such as curvature, shear deformation, and stiffness.

Chapter 5 presented a numerical model to simulate the experimental results. A method based on moment-curvature relationships obtained from generalized stress-strain curves for RC, HPFRC, and steel was adopted in the simulation. The assumptions made for the curvature distribution in the plastic hinge regions of beams, and for deformations due to shear and bond-slip, led to reasonable estimates of the force-displacement envelopes. Nonlinear seismic analyses of single-degree-of-freedom systems indicated that the displacement calculated for systems representing the beams reinforced with Grade-97 bars were about 10% larger than the displacement calculated for systems representing the beams reinforced with Grade-60 bars.

Appendix A described the material properties, specimen dimensions and instrumentation, test setup, data acquisition, and test procedure. The measured compressive strength of concrete in the specimens ranged from 5800 psi to 6400 psi. The measured steel properties for the Grade-60 bars were \( f_y = 65 \) ksi, \( \varepsilon_{su} = 15\% \), and \( f_u / f_y = 1.5 \); for the Grade-97 bars, \( f_y = 97 \) ksi, \( \varepsilon_{su} = 10\% \), and \( f_u / f_y = 1.2 \). The differences in ultimate elongation of the bars, \( \varepsilon_{su} \), and in the ratio of tensile strength-to-yield strength, \( f_u / f_y \), did not severely affect the strength and deformation capacity of the specimens reinforced with Grade-97 bars.
6.2 Conclusions

Based on the test results and analyses presented, within the domain of parameters defined in Chapter 3, two sets of conclusions are drawn regarding the use of ultrahigh strength steel bars (Grade 97) as concrete reinforcement. The first set relates to reinforced concrete (RC) beams without fibers, and the second set relates to high performance fiber reinforced concrete beams (HPFRC).

6.2.1 RC beams:

(1) On Flexural Strength. Replacing conventional Grade 60 longitudinal bars with reduced amounts of Grade-97 bars maintained the flexural strength. A reasonable estimate of the flexural strength of concrete sections reinforced with Grade-97 bars were obtained by using a simplified method of analysis based on the traditional assumption of a rectangular stress block for the concrete and an elastoplastic characterization for the steel bars.

(2) On Crack Width. For service load conditions (assumed to occur at about 60% of the nominal flexural strength), the maximum width of flexural cracks in the RC specimens reinforced with Grade-60 bars did not exceed 0.02 in., whereas for the RC specimens reinforced with Grade-97 bars the maximum crack widths reached 0.035 in. For the test beams having the same number of bars, the crack widths were proportional to the specified yield strength of the longitudinal reinforcement.

(3) On Deformation Capacity. Replacing conventional ASTM A706 Grade 60 longitudinal bars with reduced amounts of Grade-97 bars maintained the deformation capacity in excess of 10% drift ratio. This level of deformation far exceeds what would be expected in modern building structures subjected to strong ground motions.
(4) **On Hysteresis.** Reducing the amount of longitudinal reinforcement and increasing the yield strength of the reinforcement reduced the post-cracking stiffness and increased the yield deformation, leading to a reduction of the area enclosed by the force-displacement hysteresis loops. The secant stiffness to first-yield of the longitudinal reinforcement for the test beams with Grade-60 bars was nearly 30% higher than for the test beams with Grade-97 bars.

(5) **On Nonlinear Seismic Response.** Nonlinear seismic analyses of SDOF systems indicated that the maximum displacement calculated for systems representing RC beams with Grade-97 bars were, on average, less than 10% larger than the displacement calculated for systems representing RC beams with Grade-60 bars.

6.2.2 **For HPFRC beams:**

(6) **On Flexural Strength.** For beam specimens with the same longitudinal reinforcement, HPFRC beams reached flexural strengths as high as 5/4 the strength of the RC beams. Although the peak strength for the HPFRC specimens was attained at drift ratios between 2 and 3%, the flexural strengths beyond 3% drifts were nearly identical for the HPFRC and RC specimens.

(7) **On Crack Width.** For service load conditions (assumed to occur at about 60% of the nominal flexural strength), the maximum width of flexural cracks in the HPFRC specimens reinforced with Grade-97 bars did not exceed 0.02 in., whereas for the RC specimens reinforced with Grade-97 bars the maximum crack widths reached 0.035 in. The maximum widths of flexural and shear cracks of the HPFRC specimens reinforced with Grade-97 bars were generally smaller than the maximum crack widths of the RC specimens reinforced with Grade-60 bars.

(8) **On Deformation Capacity.** Replacing conventional (Grade 60) longitudinal bars in RC beams with reduced amounts of Grade-97 bars in HPFRC beams, maintained the deformation
capacity in excess of 10% drift ratio. The flexural strength of the HPFRC beams remained within 20% of the peak strength through 10% drift, even after crack localization initiated at drift ratios as low as 2%.

(9) **On Hysteresis.** The stiffness reduction associated with increased cyclic deformation was less pronounced in the HPFRC beams than in the RC beams. For beams reinforced with Grade-97 bars, the unloading stiffness in the shear-drift curve was about 20% higher in the HPFRC beams over the RC beams. The unloading stiffness for the HPFRC beams with Grade-97 bars was about 10% higher than for the RC beams with Grade-60 bars.

(10) **On Shear Strength.** Reducing the amount of transverse reinforcement (Grade 60) in HPFRC beams to one-half the amount used in RC beams, by increasing the spacing of the hoops from $d/4$ to $d/2$, did not affect the deformation capacities of the HPFRC beams. The inferred shear force contribution of HPFRC was as high as $V_c = 4\sqrt{f_{c'}} (\text{psi}) \, bd$. 
TABLES
Table 2.1 – ASTM Standards for Concrete Reinforcement in 1959

<table>
<thead>
<tr>
<th>ASTM Designation</th>
<th>Type of Steel</th>
<th>Specified Yield Strength, $f_y$ (ksi)</th>
<th>Specified Tensile Strength, $f_u$ (ksi)</th>
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</thead>
<tbody>
<tr>
<td>A432 – 59T</td>
<td>Billet</td>
<td>60</td>
<td>90</td>
</tr>
<tr>
<td>A16 – 59T</td>
<td>Rail</td>
<td>60</td>
<td>90</td>
</tr>
<tr>
<td>A431 – 58T</td>
<td>Billet</td>
<td>75</td>
<td>100</td>
</tr>
</tbody>
</table>

1 As reported by Hognestad (1961)
Table 3.1 – Specimen Description

<table>
<thead>
<tr>
<th>#</th>
<th>Specimen</th>
<th>Longitudinal Reinforcement</th>
<th>Transverse Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Top</td>
<td>Bottom</td>
</tr>
<tr>
<td>1</td>
<td>CC4-X</td>
<td>4#7</td>
<td>4#7</td>
</tr>
<tr>
<td>2</td>
<td>UC4-X</td>
<td>4#6$^4$</td>
<td>4#6$^4$</td>
</tr>
<tr>
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<td>UC4-F</td>
<td>4#6$^4$</td>
<td>4#6$^4$</td>
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<td>4#6$^4$</td>
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<td>CC4-X$^*$</td>
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<td>4#7</td>
</tr>
<tr>
<td>7</td>
<td>UC2-F$^*$</td>
<td>2#6$^4$</td>
<td>4#6$^4$</td>
</tr>
</tbody>
</table>

1. See Table A.1 for concrete properties per test specimen.
2. Transverse reinforcement consists of #3 rectangular hoops.
3. Specified $f_y$. For measured $f_y$, see Table A.3.
4. Actual bar diameter is 0.70 in. with an area of 0.39 in$^2$. 
Table 3.2 – Loading Protocol

<table>
<thead>
<tr>
<th>Step&lt;sup&gt;1&lt;/sup&gt;</th>
<th>Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.15</td>
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<tr>
<td>2</td>
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</tr>
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<td>3</td>
<td>0.30</td>
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<td>0.40</td>
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</tr>
<tr>
<td>11</td>
<td>4.0</td>
</tr>
<tr>
<td>12</td>
<td>5.0</td>
</tr>
</tbody>
</table>

<sup>1</sup> Two cycles of loading in each step, following recommendations in FEMA 461 (2007).
Table 4.1 – Maximum Measured Drift and Shear

<table>
<thead>
<tr>
<th>#</th>
<th>Specimen</th>
<th>Maximum Drift, %</th>
<th>Maximum Shear, $V_{max}$ kip</th>
<th>$v_{max} = \frac{V_{max}}{b \cdot d \sqrt{f_c}}$</th>
<th>$v_c = \frac{V_{max} - V_s}{b \cdot d \sqrt{f_c}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>CC4-X</td>
<td>&gt; 15%</td>
<td>55</td>
<td>5.5</td>
<td>0</td>
</tr>
<tr>
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<td>UC4-X</td>
<td>&gt; 15%</td>
<td>53</td>
<td>5.3</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>UC4-F</td>
<td>12%</td>
<td>64</td>
<td>6.3</td>
<td>0.4</td>
</tr>
<tr>
<td>4</td>
<td>UC2-F</td>
<td>11%</td>
<td>61</td>
<td>6.2</td>
<td>3.2</td>
</tr>
<tr>
<td>5</td>
<td>CC2-F</td>
<td>&gt;15%</td>
<td>69</td>
<td>7.1</td>
<td>4.0</td>
</tr>
<tr>
<td>6</td>
<td>CC4-X$</td>
<td>&gt;15%</td>
<td>56</td>
<td>5.6</td>
<td>0</td>
</tr>
<tr>
<td>7</td>
<td>UC2-F$</td>
<td>11%</td>
<td>60</td>
<td>6.0</td>
<td>3.0</td>
</tr>
</tbody>
</table>

1 Based on $f_c'$ (psi) at test date (see Table A.1), $b = 16$ in., and $d = 8$ in.
2 $v_c$ is the shear stress attributed to concrete assuming the contribution of transverse steel reinforcement is given by $V_s = A_v f_{yt} d / s$, where $A_v = 0.22$ in$^2$, $f_{yt} = 68$ ksi (see Table A.3), and $d / s = 2$ or 4
Table 5.1 – Assumed Material Properties of Concrete in Compression

<table>
<thead>
<tr>
<th>Material</th>
<th>$f'_c$, psi</th>
<th>$E_c$, ksi</th>
<th>$\epsilon_0$</th>
<th>$z$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined Concrete</td>
<td>6000</td>
<td>4400</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Confined Concrete</td>
<td>6000</td>
<td>4400</td>
<td>0.0024</td>
<td>20</td>
</tr>
<tr>
<td>Fiber Reinforced Concrete (HPFRC)</td>
<td>6000</td>
<td>4400</td>
<td>0.0024</td>
<td>20</td>
</tr>
</tbody>
</table>

Table 5.2 – Calculated and Measured Yield Moment

<table>
<thead>
<tr>
<th>#</th>
<th>Specimen</th>
<th>Calculated Moment, k-ft</th>
<th>Measured Moment, k-ft</th>
<th>Measured/Calculated</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>CC4-X</td>
<td>87</td>
<td>90</td>
<td>1.03</td>
</tr>
<tr>
<td>2</td>
<td>UC4-X</td>
<td>88</td>
<td>87</td>
<td>0.99</td>
</tr>
<tr>
<td>3</td>
<td>UC4-F</td>
<td>117</td>
<td>117</td>
<td>1.00</td>
</tr>
<tr>
<td>4</td>
<td>UC2-F</td>
<td>117</td>
<td>111</td>
<td>0.95</td>
</tr>
<tr>
<td>5</td>
<td>CC2-F</td>
<td>119</td>
<td>102</td>
<td>0.86</td>
</tr>
<tr>
<td>6</td>
<td>CC4-X$</td>
<td>88</td>
<td>89</td>
<td>1.01</td>
</tr>
<tr>
<td>7</td>
<td>UC2-F$</td>
<td>117</td>
<td>113</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td>Mean</td>
<td></td>
<td></td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td>Standard Deviation</td>
<td></td>
<td></td>
<td>0.06</td>
</tr>
</tbody>
</table>
Table 5.3 – Calculated and Measured Maximum Moment

<table>
<thead>
<tr>
<th></th>
<th>Specimen</th>
<th>Calculated Moment, k-ft</th>
<th>Measured Moment, k-ft</th>
<th>Measured/Calculated</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>CC4-X</td>
<td>116</td>
<td>109</td>
<td>0.94</td>
</tr>
<tr>
<td>2</td>
<td>UC4-X</td>
<td>91</td>
<td>105</td>
<td>1.13</td>
</tr>
<tr>
<td>3</td>
<td>UC4-F</td>
<td>117</td>
<td>128</td>
<td>1.06</td>
</tr>
<tr>
<td>4</td>
<td>UC2-F</td>
<td>1117</td>
<td>123</td>
<td>1.02</td>
</tr>
<tr>
<td>5</td>
<td>CC2-F</td>
<td>127</td>
<td>137</td>
<td>1.08</td>
</tr>
<tr>
<td>6</td>
<td>CC4-X$</td>
<td>100</td>
<td>112</td>
<td>1.12</td>
</tr>
<tr>
<td>7</td>
<td>UC2-F$</td>
<td>117</td>
<td>121</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Mean 1.07
Standard Deviation 0.07
<table>
<thead>
<tr>
<th>#</th>
<th>Specimen</th>
<th>Calculated, %</th>
<th>Measured, %</th>
<th>Measured/Calculated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Flexure</td>
<td>Slip</td>
<td>Shear</td>
</tr>
<tr>
<td>1</td>
<td>CC4-X</td>
<td>0.53</td>
<td>0.27</td>
<td>0.13</td>
</tr>
<tr>
<td>2</td>
<td>UC4-X</td>
<td>0.76</td>
<td>0.40</td>
<td>0.19</td>
</tr>
<tr>
<td>3</td>
<td>UC4-F</td>
<td>0.84</td>
<td>0.35</td>
<td>0.19</td>
</tr>
<tr>
<td>4</td>
<td>UC2-F</td>
<td>0.84</td>
<td>0.35</td>
<td>0.19</td>
</tr>
<tr>
<td>5</td>
<td>CC2-F</td>
<td>0.59</td>
<td>0.27</td>
<td>0.14</td>
</tr>
<tr>
<td>6</td>
<td>CC4-X$</td>
<td>0.54</td>
<td>0.27</td>
<td>0.13</td>
</tr>
<tr>
<td>7</td>
<td>UC2-F$</td>
<td>0.84</td>
<td>0.35</td>
<td>0.19</td>
</tr>
</tbody>
</table>

1 Strain gage in the longitudinal bar at the critical section was not installed or malfunctioned.
Table 5.5 – Calculated (Simplified Method) and Measured Yield Moment

<table>
<thead>
<tr>
<th>#</th>
<th>Specimen</th>
<th>Calculated Moment, k-ft</th>
<th>Measured Moment, k-ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>CC4-X</td>
<td>85</td>
<td>90</td>
</tr>
<tr>
<td>2</td>
<td>UC4-X</td>
<td>89</td>
<td>87</td>
</tr>
<tr>
<td>6</td>
<td>CC4-XS</td>
<td>85</td>
<td>89</td>
</tr>
</tbody>
</table>

Table 5.6 – Calculated (Simplified Method) and Measured Maximum Moment

<table>
<thead>
<tr>
<th>#</th>
<th>Specimen</th>
<th>Calculated Moment, k-ft</th>
<th>Measured Moment, k-ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>CC4-X</td>
<td>103</td>
<td>109</td>
</tr>
<tr>
<td>2</td>
<td>UC4-X</td>
<td>108</td>
<td>105</td>
</tr>
<tr>
<td>6</td>
<td>CC4-XS</td>
<td>104</td>
<td>112</td>
</tr>
</tbody>
</table>
### Table 5.7 – Properties of SDOF Systems Considered

<table>
<thead>
<tr>
<th>No.</th>
<th>System(^1)</th>
<th>Period of Vibration(^2), T</th>
<th>Spectral Acceleration Coefficient(^3), S(_A)</th>
<th>Strength Ratio(^4), SR</th>
<th>Yield Strength Coefficient(^5), C(_y)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Grade 60</td>
<td>0.60</td>
<td>1.00</td>
<td>1/3</td>
<td>0.33</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>0.90</td>
<td>0.83</td>
<td>1/3</td>
<td>0.28</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>1.20</td>
<td>0.63</td>
<td>1/3</td>
<td>0.21</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>0.67</td>
<td>1.00</td>
<td>1/6</td>
<td>0.17</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>1.01</td>
<td>0.75</td>
<td>1/6</td>
<td>0.14</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>1.34</td>
<td>0.56</td>
<td>1/6</td>
<td>0.10</td>
</tr>
<tr>
<td>7</td>
<td>Grade 97</td>
<td>0.67</td>
<td>1.00</td>
<td>1/2.7</td>
<td>0.28</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>1.01</td>
<td>0.75</td>
<td>1/5.4</td>
<td>0.14</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td>1.34</td>
<td>0.56</td>
<td>1/5.4</td>
<td>0.10</td>
</tr>
</tbody>
</table>

1 The Grade-97 systems represent equivalent alternatives to the Grade-60 systems. Both the Grade-97 and Grade-60 systems target identical strength. However, the stiffness of the Grade-97 systems is 0.80 times the stiffness of the Grade-60 systems. In all cases, the post-yield stiffness was defined as 5% of the initial stiffness.

2 Target periods of vibration for the Grade-60 systems are set to 0.6, 0.9, and 1.2 s for a unit mass, from which the stiffness is derived. The stiffness of the Grade-97 systems is 0.80 times that of the Grade-60 systems.

3 Linear-response acceleration (divided by g) of a 5-percent damped SDOF system of period T. Defined using S\(_A\) = S\(_{D1}\) / T ≤ S\(_{DS}\), where S\(_{DS}\) = 1.0 and S\(_{D1}\) = 0.75.

4 Yield force, V\(_y\), of nonlinear SDOF system (of initial period T) divided by the force induced in a linear SDOF system of period T.

5 C\(_y\) = S\(_A\)·SR = V\(_y\) / W, where SR is the strength ratio and S\(_A\) is the spectral acceleration coefficient obtained from the design spectrum for a system of period T. The value of C\(_y\) corresponds to the yield strength, V\(_y\), divided by the weight, W, of the SDOF system.
Table 5.8 – Ground Motions Considered

<table>
<thead>
<tr>
<th>Station1</th>
<th>Earthquake</th>
<th>Magnitude</th>
<th>Epicentral Distance (km)</th>
<th>Site Class2</th>
<th>PGA3 (g)</th>
<th>PGV4 (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Berkeley, NS (Station 58471) Lawrence Berkeley Lab, Calif.</td>
<td>Loma Prieta 10-17-1989</td>
<td>7.1</td>
<td>99.0</td>
<td>C</td>
<td>0.117</td>
<td>22.0</td>
</tr>
<tr>
<td>Beverly Hills, NS (Station 00013) 14145 Mulholland Dr., Calif.</td>
<td>Northridge 01-17-1994</td>
<td>6.1</td>
<td>12.7</td>
<td>D</td>
<td>0.443</td>
<td>59.3</td>
</tr>
<tr>
<td>El Centro, NS (Station 117) Imperial Valley Irrigation District, Calif.</td>
<td>Imperial Valley 05-18-1940</td>
<td>6.9</td>
<td>16.9</td>
<td>D</td>
<td>0.348</td>
<td>33.2</td>
</tr>
<tr>
<td>El Centro, NS (Station 01335) Imperial Co. Center Grounds, Calif.</td>
<td>Superstition Hills 11-24-1987</td>
<td>6.6</td>
<td>36.0</td>
<td>D</td>
<td>0.341</td>
<td>46.6</td>
</tr>
<tr>
<td>Lake Hughes, N21E (Station 125, File 1) Fire Station #78, Calif.</td>
<td>San Fernando 02-09-1971</td>
<td>7.5</td>
<td>31.3</td>
<td>C</td>
<td>0.148</td>
<td>18.5</td>
</tr>
<tr>
<td>Lancaster, NS (Station 24475) Fox Airfield Grounds, Calif.</td>
<td>Northridge 01-17-1994</td>
<td>6.1</td>
<td>66.0</td>
<td>D</td>
<td>0.064</td>
<td>5.44</td>
</tr>
<tr>
<td>Los Angeles, NS (Station 24303) Hollywood Storage Building Grounds, Calif.</td>
<td>Northridge 01-17-1994</td>
<td>6.1</td>
<td>23.0</td>
<td>D</td>
<td>0.231</td>
<td>18.2</td>
</tr>
<tr>
<td>Richmond, S10E (Station 58505) City Hall Parking Lot, Calif.</td>
<td>Loma Prieta 10-17-1989</td>
<td>7.1</td>
<td>108.0</td>
<td>D</td>
<td>0.106</td>
<td>14.7</td>
</tr>
<tr>
<td>Santa Barbara, S48E (Station 283) Courthouse, Calif.</td>
<td>Kern County 07-21-1952</td>
<td>7.5</td>
<td>87.8</td>
<td>C</td>
<td>0.131</td>
<td>19.3</td>
</tr>
<tr>
<td>Wrightwood, NS (Station 23590) Jackson Flat, Calif.</td>
<td>Northridge 01-17-1994</td>
<td>6.1</td>
<td>76.0</td>
<td>C</td>
<td>0.056</td>
<td>5.06</td>
</tr>
</tbody>
</table>

1 Center for Engineering Strong Motion Data, CESMD (2011). Sensors are part of the California Strong Motion Instrumentation Program (CSMIP).
2 Based on the classifications of sites in ASCE/SEI 7-10 (2010).
3 PGA: Peak ground acceleration.
4 PGV: Peak ground velocity.
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b) Specimens 4, 5
c) Specimen 6

d) Specimen 7

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b) Schematic View of the Location of Linear Potentiometers

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At the Start of Test
At 1% Drift Ratio

At 2% Drift Ratio
At 3% Drift Ratio

At 4% Drift Ratio
At 5% Drift Ratio

Figure 4.1 – Specimen CC4-X during Test
Figure 4.2 – Specimen CC4-X at the End of Test

Figure 4.3 – Specimen CC4-X after Removal of Loose Concrete (North Beam)
Figure 4.4 – Specimen UC4-X during Test
Figure 4.5 – Specimen UC4-X at the End of Test

Figure 4.6 – Specimen UC4-X after Removal of Loose Concrete (North Beam)
At the Start of Test

At 1% Drift Ratio

At 2% Drift Ratio

At 3% Drift Ratio

At 4% Drift Ratio

At 5% Drift Ratio

Figure 4.7 – Specimen UC4-F during Test
Figure 4.8 – Specimen UC4-F at the End of Test
Figure 4.9 – Specimen UC2-F during Test
Figure 4.10 – Specimen UC2-F at the End of Test
At the Start of Test

At 1% Drift Ratio

At 2% Drift Ratio

At 3% Drift Ratio

At 4% Drift Ratio

At 5% Drift Ratio

Figure 4.11 – Specimen CC2-F during Test
Figure 4.12 – Specimen CC2-F at the End of Test
Figure 4.13 – Specimen CC4-X$ during Test
Figure 4.14 – Specimen CC4-X$ at the End of Test

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Figure 4.16 – Specimen UC2-F$ during Test
Figure 4.17 – Specimen UC2-F$ at the End of Test
Figure 4.18 – Shear vs. Drift Ratio for Specimen CC4-X, South Beam

Figure 4.19 – Shear vs. Drift Ratio for Specimen CC4-X, North Beam
Figure 4.20 – Shear vs. Drift Ratio for Specimen UC4-X, South Beam

Figure 4.21 – Shear vs. Drift Ratio for Specimen UC4-X, North Beam
Figure 4.22 – Shear vs. Drift Ratio for Specimen UC4-F, South Beam

Figure 4.23 – Shear vs. Drift Ratio for Specimen UC4-F, North Beam
Figure 4.24 – Shear vs. Drift Ratio for Specimen UC2-F, South Beam

Figure 4.25 – Shear vs. Drift Ratio for Specimen UC2-F, North Beam
Figure 4.26 – Shear vs. Drift Ratio for Specimen CC2-F, South Beam

Figure 4.27 – Shear vs. Drift Ratio for Specimen CC2-F, North Beam
Figure 4.28 – Shear vs. Drift Ratio for Specimen CC4-X$, South Beam

Figure 4.29 – Shear vs. Drift Ratio for Specimen CC4-X$, North Beam
Figure 4.30 – Shear vs. Drift Ratio for Specimen UC2-F$, South Beam

Figure 4.31 – Shear vs. Drift Ratio for Specimen UC2-F$, North Beam
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Figure 4.33 – Shear vs. Drift Ratio at the Final Push for Specimen UC4-X, South Beam
Figure 4.34 – Shear vs. Drift Ratio at the Final Push for Specimen UC4-F, South Beam

Figure 4.35 – Shear vs. Drift Ratio at the Final Push for Specimen UC2-F, South Beam
Figure 4.36 – Shear vs. Drift Ratio at the Final Push for Specimen CC2-F, South Beam

Figure 4.37 – Shear vs. Drift Ratio at the Final Push for Specimen CC4-X$, South Beam
Figure 4.38 – Shear vs. Drift Ratio at the Final Push for Specimen UC2-F$, South Beam
Figure 4.39 – Envelope of Shear vs. Drift Ratio for Specimen CC4-X

Figure 4.40 – Envelope of Shear vs. Drift Ratio for Specimen UC4-X
Figure 4.41 – Envelope of Shear vs. Drift Ratio for Specimen UC4-F

Figure 4.42 – Envelope of Shear vs. Drift Ratio for Specimen UC2-F
Figure 4.43 – Envelope of Shear vs. Drift Ratio for Specimen CC2-F

Figure 4.44 – Envelope of Shear vs. Drift Ratio for Specimen CC4-X$
Figure 4.45 – Envelope of Shear vs. Drift Ratio for Specimen UC2-F$
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Figure 4.46 – Continued
Figure 4.47 – Measured Moment-Curvature Relationships for Specimen UC4-X
Figure 4.47 – Continued
Figure 4.48 – Measured Moment-Curvature Relationships for Specimen UC4-F
Figure 4.49 – Measured Moment-Curvature Relationships for Specimen UC2-F
Figure 4.50 – Measured Moment-Curvature Relationships for Specimen CC2-F
Figure 4.51 – Measured Moment-Curvature Relationships for Specimen CC4-X$
Figure 4.51 – Continued
Figure 4.52 – Measured Moment-Curvature Relationships for Specimen UC2-F$
Figure 4.52 – Continued
Figure 4.53 – Measured Longitudinal Strains vs. Drift Ratio for Specimen CC4-X
Figure 4.53 – Continued
Figure 4. 53 – Continued
Figure 4.54 – Measured Longitudinal Strains vs. Shear for Specimen CC4-X
Figure 4.54 – Continued
Figure 4.54 – Continued
Figure 4.54 – Continued
Figure 4.55 – Measured Longitudinal Strains vs. Drift Ratio for Specimen UC4-X
Figure 4.55 – Continued
Figure 4.56 – Measured Longitudinal Strains vs. Shear for Specimen UC4-X
Figure 4.56 – Continued
Figure 4.56 – Continued
Figure 4.57 – Measured Longitudinal Strains vs. Drift Ratio for Specimen UC4-F
Figure 4.57 – Continued
Figure 4.57 – Continued
Figure 4.58 – Measured Longitudinal Strains vs. Shear for Specimen UC4-F
Figure 4. 58 – Continued
Figure 4.59 – Measured Longitudinal Strains vs. Drift Ratio for Specimen UC2-F
Figure 4.59 – Continued
Figure 4.59 – Continued
Figure 4.60 – Measured Longitudinal Strains vs. Shear for Specimen UC2-F
Figure 4.60 – Continued
Figure 4. 60 – Continued
Figure 4.61 – Measured Longitudinal Strains vs. Drift Ratio for Specimen CC2-F
Figure 4.62 – Measured Longitudinal Strains vs. Shear for Specimen CC2-F
Figure 4.63 – Measured Longitudinal Strains vs. Drift Ratio for Specimen CC4-X$
Figure 4. 63 – Continued
Figure 4.6 – Continued
Figure 4.64 – Measured Longitudinal Strains vs. Shear for Specimen CC4-X$
Figure 4.64 – Continued
Figure 4.64 – Continued
Figure 4. 64 – Continued
Figure 4.65 – Measured Longitudinal Strains vs. Drift Ratio for Specimen UC2-F$
Figure 4.6 – Continued
Figure 4.65 – Continued
Figure 4.66 – Measured Longitudinal Strains vs. Shear for Specimen UC2-F$
Figure 4.6 – Continued
Figure 4.6 – Continued
Figure 4.67 – Measured Hoop Strains vs. Drift Ratio for Specimen CC4-X
Figure 4.67 – Continued
Figure 4.67 – Continued
Figure 4.68 – Measured Hoop Strains vs. Shear for Specimen CC4-X
Figure 4.68 – Continued
Figure 4.68 – Continued
Figure 4.69 – Measured Hoop Strains vs. Drift Ratio for Specimen UC4-F
Figure 4.69 – Continued
Figure 4.70 – Measured Hoop Strains vs. Shear for Specimen UC4-F
Figure 4.70 – Continued
Figure 4.71 – Measured Hoop Strains vs. Drift Ratio for Specimen UC2-F
Figure 4.71 – Continued
Figure 4.72 – Measured Hoop Strains vs. Shear for Specimen UC2-F
Figure 4.72 – Continued
Figure 4.73 – Measured Hoop Strains vs. Drift Ratio for Specimen CC2-F

Figure 4.74 – Measured Hoop Strains vs. Shear for Specimen CC2-F
Figure 4.75 – Measured Hoop Strains vs. Drift Ratio for Specimen CC4-X$
Figure 4.75 – Continued
Figure 4.75 – Continued
Figure 4.76 – Measured Hoop Strains vs. Shear for Specimen CC4-X$
Figure 4.7 – Continued
Figure 4.76 – Continued
Figure 4.77 – Measured Hoop Strains vs. Drift Ratio for Specimen UC2-F$
Figure 4.7 – Continued
Figure 4.78 – Measured Hoop Strains vs. Shear for Specimen UC2-F$
Figure 4.7 – Continued
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Figure 4.82 – Measured Cracks Widths at Peaks of Negative Drifts for Specimen UC4-X
Figure 4.83 – Measured Cracks Widths at Peaks of Positive Drifts for Specimen UC4-F

Figure 4.84 – Measured Cracks Widths at Peaks of Negative Drifts for Specimen UC4-F
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$
$\alpha = 0.43$
$R^2 = 0.9662$

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$\alpha = 0.51$
$R^2 = 0.9997$
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\[ f_y = 65 \text{ ksi} \]
\[ f_u = 98 \text{ ksi} \]
\[ \epsilon_{su} = 0.10 \]
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$
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\[ K_u = K_y \left( \frac{D_y}{D_{max}} \right)^\alpha \]
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Hysteresis Parameters:
\[ V_y = 42 \text{ kip} ; D_y = 1.18\% \]
\[ K_{py} = 0.05K_y ; \alpha = 0.4 \]

Figure 5.33 – Cyclic Response of Specimen UC4-X, Takeda Hysteresis Model vs. Measured Response

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APPENDIX A

EXPERIMENTAL WORK

A.1 Introduction

This Appendix covers the information about material properties, dimensions of the test specimens, and design of the test setup. The instrumentation of the specimens, data acquisition, and the test procedure are also described here.

A.2 Materials

A.2.1 Concrete

Concrete for all specimens was provided by Centre Concrete, a ready mix supplier in State College, Pennsylvania. Specimens were cast in three separate sessions, resulting in three different concrete batches. The specified 28-day compressive strength of concrete for all batches was 6000 psi. Designations for test specimens, concrete batch used for each specimen, and the concrete age and strength at the time of test are summarized in Table A.1. The same mix proportions, presented in Table A.2, was used for all specimens. Higher dosage of water reducing agent was used for specimens with fiber to increase the workability of concrete. The maximum aggregate size was limited to 1/2 in., to ease the mixing of fibers in the concrete. As indicated in Table A.1, the compressive strengths of the concrete for the three batches were within 10% of the specified strength of 6000 psi.
Compressive strength of concrete was measured by conducting compressive tests on 4-in. diameter by 8-in. long cylinders with neoprene-padded steel end caps. The rate of strength gain for each batch is presented in Figure A.1, Figure A.2, and Figure A.3. All batches gained more than 50% of their ultimate strength in the first 3 days after casting. Higher ambient temperature in July is very likely the reason for the higher rate of early strength gain (75% in 3 days) for batch 3.

Batches 2 and 3 contained fibers. The fibers were added to the concrete inside the delivery truck before casting. Therefore, it was possible to cast sample cylinders of the same batch with and without fibers. A comparison of the average 28-day compressive strength of these samples is shown in Figure A.4. There was a slight increase in the compressive strength of samples made of High Performance Fiber Reinforced Concrete (HPFRC) as compared to the ones without fibers (RC).

A.2.2 Reinforcement

Reinforcement used to fabricate the hoops for all specimens was made of #3 ASTM A706 (2006) grade 60 steel. All #3 bars came from the same shipment with the same mill certificate. Three random samples were chosen, and tested to determine the yield and ultimate strength (Table A.3).

For longitudinal bars, two different types of steel were used. All specimens identified by the letter “C” at the start of their designation (CC4-X, CC2-F, and CC4-X$) used #7 ASTM A706 Grade-60 deformed steel bars. All other specimens identified by the letter “U” at the start of their designation (UC4-X, UC4-F, UC2-F, and UC2-F$) used nominal #6 Grade-97 bars. The #6 refers to a metric $\Phi18$ bar supplied by SAS Stresssteel, Inc. (http://www.annahuette.com). Three random samples for each type of longitudinal bar were tested to determine the stress-strain curve of the steel reinforcement. The curves obtained are presented in Figure A.5 and Figure A.6. The
strain in a sample bar was measured by means of an extensometer, which was removed at about 2% strain. The ultimate elongation of the bars after fracture, in an 8-in. gage length, was measured and reported as the ultimate strain ($\varepsilon_{uu}$ in Table A.3). For the #6 bars, the 0.2% offset method was used for determining the yield strength, as suggested by ASTM A370 (2006), see Table A.3 and Figure A.7. The #6 (Grade 97) bars had an average yield strength of $f_y = 97.4$ ksi, an average tensile strength of $f_u = 117$ ksi, and an elongation (8-in. gage length) of $\varepsilon_{uu} = 10.4\%$. It is important to note that the ratio $f_u / f_y = 1.20$, is lower than the minimum of 1.25 required for seismic applications according to Chapter 21 of ACI 318-08 (2008).

A.3 Specimens

Test specimens consisted of a central joint with two beams spanning in opposite directions. Each beam element was intended to represent a cantilever beam with the central joint acting as the base of the cantilever. Each beam was supported by a roller 2 ft. away from the face of the central joint. The total length of the specimen provided sufficient development length for the longitudinal reinforcement. The central stub of the specimen was loaded by a hydraulic jack. A schematic front view of the test frame and the test specimen is shown in Figure A.8. Typical setup before the start of the test is shown in Figure 3.3.

A.3.1 As-Built Dimensions

The overall dimensions are shown in Figure 3.1. However, due to imperfections the final dimensions were slightly off. The as-built dimensions identified in Figure A.9 are reported in Table A.4 and Table A.5.
A.3.2 Reinforcement Details

The reinforcing details of the specimens are shown in Figure 3.2. All beams made with conventional steel (Grade 60) had a reinforcement ratio, $\rho$, of 1.9%, while the beams made with ultrahigh strength steel (Grade 97) had a reinforcement ratio of 1.2%.

Transverse reinforcement was spaced at the minimum of $d/4$ or $6d_b$ (where $d/4$ controlled for all specimens). They were proportioned to satisfy the shear strength requirements based on $M_{pr}$ and $V_c = 0$, following Chapter 21 of ACI 318-08 (2008). For the HPFRC specimens, spacing of $d/2$ was also investigated. The transverse steel reinforcement ratio, $\rho_t$, was 0.69% for concrete (RC) specimens and 0.34% for the HPFRC specimens. The transverse reinforcement was fabricated as hoops with seismic hooks at both ends in accordance with Chapter 21 of ACI-318-08 (2008). The transverse reinforcement details are presented in Figure A.10. A sample hoop conforming to these details is shown in Figure A.11.

The length of the specimens was sufficient to develop the reinforcing bars and therefore the longitudinal reinforcement was not hooked at the ends. Although the Grade-97 bar was threaded, its bond characteristics were similar to that of conventional bars (CTL Group, 2005).

A.3.3 Formwork, Casting, and Curing

A set of four wood forms was designed and built for casting the specimens. These forms were fabricated in segments (Figure A.12) and bolted together before casting. The bolted assembly allowed repetitive use and increased the lifespan of the molds. The assembled forms are shown in Figure A.13.

The reinforcement cages shown in Figure A.14 were fabricated in the laboratory and placed in the formwork after lubrication with form oil (Figure A.15). At the time of casting,
concrete from the delivery truck was placed and vibrated in these forms (Figure A.16). For the HPFRC batches, additional water reducing agent was required to attain similar workability to that of concrete without fibers. Additional concrete was deposited in wheel barrows to cast the 4x8” cylinders and the 4x14” beams (Figure A.17). A few hours after pouring concrete all cast elements were covered with wet burlap and nylon sheets as shown in Figure A.18. For three days, at 24-hour intervals, the burlap was rewatered for proper curing of the specimen.

All concrete elements were stripped of their formwork after one week (Figure A.19). Specimens were kept in the laboratory until the time of testing. Concrete cylinders were typically tested at 3, 7, 14, and 28 days. At the time of test, additional cylinders were tested to obtain the compressive strength of the concrete (Figure A.20).

A.4 Test Procedure

The experiments were designed to investigate the nonlinear behavior of concrete beams reinforced with ultrahigh strength steel. The load history consisted of large cyclic deformations representative of the effects induced during strong seismic events (Table 3.2 and Figure 3.4). A loading frame was designed and built to apply the loading protocol.

A.4.1 Test Setup

It was estimated that a maximum force of 120 kips was required to run the tests. Since there was no strong floor available in the Architectural Engineering (AE) Laboratory at Penn State, it was decided to design and build a vertical self-reacting loading frame. To extend the usage of the loading frame for future needs, it was designed for a load capacity of 240 kips. In addition to the extra loading capacity, dimensions of the frame were also enlarged to the extent
allowed by the space limitations in the AE Laboratory (Figure A.8). Extra holes were drilled in
the top and bottom beams of the loading frame to accommodate variable configurations.

The loading frame components were fabricated by Miller Welding Service, a local steel
shop in State College, Pennsylvania. These parts were then shipped to the AE Laboratory and
assembled using bolted connections.

The main parts of the frame consisted of two W24X131 beams (Figure A.21 and Figure
A.22), and two HSS6.000X0.250 columns (Figure A.23).

In addition to the main frame components, supports and loading plates were also
designed and fabricated. The accessory parts included two top flange connection plates (Figure
A.24), two bottom supports (Figure A.25), four support plates and rollers (Figure A.26), two top
supports (Figure A.27), one bottom stub plate (Figure A.28), and one top stub plate (Figure
A.29). All of the accessories were installed by bolts, nuts, and threaded rods.

Specimens were loaded through the center stub using an Enerpac RR-3006 double acting
cylinder. The cylinder was attached to the steel reaction frame as shown in Figure A.8. The
required hydraulic pressure for the cylinder was provided by an electric pump manually
controlled by the operator. To avoid applying a moment at the central stub and to provide a
rotational degree of freedom at the center stub, the loading cylinder was connected to the top stub
plate by means of a pinned-connection adaptor. Details of this adaptor are shown in Figure A.30.
The adaptor was a custom-made hinge designed and built in three different parts: a top part
welded to the actuator plunger (Figure A.31); a 16-in. long roller of 3.5-in. diameter; and the
bottom part (Figure A.32) which was bolted to the top stub plate.

Each specimen was placed between the top and bottom supports as shown in Figure 3.3.
An eighth of an inch gap between the top supports and the rollers above the specimen was
provided to avoid a fixed support condition.
After aligning the specimen in the loading frame, the center stub was attached to the actuator. The displacement of the stub was controlled manually to follow the predetermined loading protocol (Table 3.2 and Figure 3.4).

A.4.2 Measurements

One of the main objectives of this investigation was to study the deformations that occur in beams reinforced with ultrahigh strength steel. To achieve this goal it was necessary to obtain the net deflection and rotation of the beams, for which all rigid motion components (rigid translations and rigid rotations) needed to be removed. Therefore, displacements at the stub face and supports were measured with respect to a fixed system of coordinates (Figure 3.5). A small math script was incorporated into the data acquisition software to calculate an adjusted drift ratio (defined below) for each beam. The applied force was plotted against the adjusted drift ratios and they were monitored during the test to correctly follow the loading protocol.

For the case of no stub rotation, the two beams would have the same drift ratio and equal to the average drift ratio, \( \delta \):

\[
\delta = \frac{\Delta}{a}
\]  

(A. 1)

where \( \Delta \) is the midspan deflection taken as the average of absolute displacement measured by the four potentiometers at the center stub and \( a \) is the shear span (24 in. for all specimens), defined as the distance between the face of the stub to the support (Figure A.33).

Due to the randomness involved in the experiment, the center stub would undergo rotations and therefore the drift in each beam shall be adjusted using the following expression:
\[ \delta_{adj} = \frac{A \pm w \beta}{a} \pm \beta \]  

(A. 2)

where \( \delta_{adj} \) is the adjusted drift, \( w \) is the distance from the centerline to the face of stub (i.e. 8 in.), and \( \beta \) is the stub rotation in radians. The rotation of the middle block was calculated based on the readings of the four potentiometers connected to opposite faces of the stub.

The other main parameter measured was the applied force. Since there was no load cell available to connect to the Enerpac Cylinder, it was originally decided to measure the applied hydraulic pressure to the actuator and calculate the load. However, after a few trials it was observed that the pressure transducers were not sufficiently accurate. The output of these transducers had a long lag after a change of direction in the applied force. Therefore, it was decided to design and build a custom-made load cell. The first option was to use the rollers at the supports and instrument them with strain gages to build load cells. This option, although advantageous because the shear in each beam would have been measured directly, was rejected due to the very small strains induced in the rollers with low signal-to-noise ratio. The second and adopted option was to use the upper part of the loading adaptor as a load cell (Figure 3.6).

The plunger of the top part of the loading adaptor was instrumented with strain gages. Strain gages used for this purpose were FLA-3-5L (from Texas Measurements, Inc.), rated for a maximum strain of 2%. Since the load adaptor was designed to be in the elastic range during all tests, the relationship between the applied force and induced strain in the plunger was linear. This custom-made load cell was calibrated against a MTS 661.23A-02 load cell (Capacity 100 kips) as shown in Figure A.34. This custom-made load cell was used in all tests to measure the applied force. Shear in each beam was obtained by dividing the applied force by two.
In addition to the above mentioned measurements, the strain of the reinforcement bars at different locations in the specimen was also captured by strain gages (Figure 3.7). In each specimen, two longitudinal bars were instrumented. On each rebar, seven strain gages were installed. The strain gages used for the longitudinal bars were YEFLA-5-5L, rated for a maximum strain of 10%. Strain gages were also applied to the transverse reinforcement: the first four hoops on each side of the stub in the specimens with a hoop spacing of $d/4$, and the first two hoops in the specimens with a hoop spacing of $d/2$. Strain gages used for the transverse reinforcement were FLA-3-5L, rated for a maximum strain of 2%. All strain gages were provided by Texas Measurements, Inc. with a resistance of 120 ohms. Due to the application of strain gages embedded in concrete, all strain gages were protected against moisture and abrasion by applying coatings recommended and provided by Texas Measurements.

The data acquisition hardware used during the tests consisted of a PCI-6021 data acquisition board coupled to an SCXI 1000 chassis by National Instruments. The chassis and other custom-made signal-conditioning equipment were connected to a personal computer to process and record sensor data.

At the peak of each loading cycle the width of cracks were measured and recorded manually with a crack comparator.

The absolute deflection of each beam has three components: shear component, bond slip, and flexural component (see Chapter 5). To measure the shear component, the front face of each beam had a grid laid out with steel contact points glued to the surface of concrete as shown in Figure A.35. The distance between points was measured at alternate peaks of the loading protocol using electronic Whittemore gages. Since the grid was made of 6-in. squares, two different Whittemore instruments with gage lengths of 6 and 8.5 in. were built and used for all tests (Figure A.36). The range of LVDTs (Linear Voltage Displacement Transducer) used in these instruments was ± 0.25 in. The recoded horizontal, vertical, and diagonal distances between gage points were
stored using the data acquisition system. The recorded data were used to calculate the shear component of beam deflections.

A.4.3 Test Steps

The following steps were followed during testing of a single specimen:

- Mount the specimen in the load frame;
- Layout grid of Whittemore reference points;
- Setup all sensors and acquire signal from each;
- Calibrate strain gages;
- Prime the face of the specimen for ease of locating and marking cracks;
- Run a small load cycle (peak less than 10 kips) to troubleshoot the setup;
- Start the loading protocol;
- Stop test at peak of each cycle and mark and measure cracks and take pictures;
- Take Whittemore readings at peaks of second cycles and at end of each step (zero force after two cycles);
- Apply two cycles per step through step #12 (each step targets a new drift ratio);
- Stop the test for final push (monotonic loading to failure) and readjust sensors;
- Run the final push;
- Finish the test and remove the loose concrete (if any) and take pictures.
Table A.1 – Concrete Properties per Test Specimen

<table>
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<tr>
<th>#</th>
<th>Specimen</th>
<th>Concrete Batch</th>
<th>Cast Date</th>
<th>Test Date</th>
<th>Compressive Strength @ Test Date, psi</th>
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<tr>
<td>1</td>
<td>CC4-X</td>
<td>1</td>
<td>12/04/2008</td>
<td>05/06/2010</td>
<td>6000</td>
</tr>
<tr>
<td>2</td>
<td>UC4-X</td>
<td>1</td>
<td>12/04/2008</td>
<td>03/29/2010</td>
<td>6200</td>
</tr>
<tr>
<td>3</td>
<td>UC4-F</td>
<td>3</td>
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<td>05/25/2010</td>
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</tr>
<tr>
<td>4</td>
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<td>3</td>
<td>07/03/2009</td>
<td>07/13/2010</td>
<td>5900</td>
</tr>
<tr>
<td>5</td>
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<td>2</td>
<td>01/29/2009</td>
<td>12/21/2009</td>
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</tr>
<tr>
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<td>1</td>
<td>12/04/2008</td>
<td>06/18/2010</td>
<td>6200</td>
</tr>
<tr>
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<td>UC2-F$</td>
<td>3</td>
<td>07/03/2009</td>
<td>07/29/2010</td>
<td>6200</td>
</tr>
</tbody>
</table>

1 Specimen designation is based on the following symbols:
CC: conventional longitudinal and conventional transverse reinforcement;
UC: ultrahigh strength longitudinal and conventional transverse reinforcement;
4: $d/s = 4$ where $d$ is effective depth and $s$ is spacing of transverse reinforcement;
2: $d/s = 2$;
X: reinforced concrete without fibers (RC specimen);
F: high performance fiber reinforced concrete (HPFRC specimen) with steel-hooked fibers (Dramix RC-80/30-BP by Bekaert) in a volume fraction of 1.5%.
$: specimen with asymmetrical reinforcement ($\rho'/\rho = 0.5$).
<table>
<thead>
<tr>
<th>Proportions</th>
<th>Units</th>
<th>Target</th>
<th>Batch 1</th>
<th>Batch 2</th>
<th>Batch 3</th>
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<tbody>
<tr>
<td>Cement</td>
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<td>747</td>
<td>710</td>
<td>726</td>
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<td>1820</td>
<td>1840</td>
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<td>Gravel&lt;sup&gt;1&lt;/sup&gt;</td>
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<td>1230</td>
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<td>1290</td>
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<td>264</td>
<td>205</td>
<td>265</td>
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<td>84</td>
<td>84</td>
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<tr>
<td>Air</td>
<td>%</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Fiber</td>
<td>Volumetric %</td>
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<td>0</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
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<td>0.40</td>
<td>0.35</td>
<td>0.29</td>
<td>0.37</td>
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<sup>1</sup> Maximum aggregate size of ½ in.
Table A.3 – Measured Steel Properties

<table>
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<tr>
<th>Reinforcing Bar Size</th>
<th>Specimen</th>
<th>Coupon</th>
<th>$f_y$, ksi</th>
<th>$\varepsilon_{fu}^1$</th>
<th>$f_u^2$, ksi</th>
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</thead>
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<sup>1</sup> Fracture elongation in 8-in. gage length
<sup>2</sup> Tensile strength
<sup>3</sup> Actual bar diameter is 0.70 in. (18 mm) with an area of 0.39 in$^2$ (250 mm$^2$)
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<sup>1</sup>All dimensions in inches. See Figure A.9 for dimension locations.
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Dimensions for Section A-A (North)

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Dimensions for Section B-B (South)

¹ All dimensions in inches. See Figure A.9 for dimension locations.
Figure A.1 – Measured Compressive Strength of Concrete versus Time, Batch 1

Figure A.2 – Measured Compressive Strength of Concrete versus Time, Batch 2
Figure A.3 – Measured Compressive Strength of Concrete versus Time, Batch 3

Figure A.4 – Average 28-Day Compressive Strength of Concrete, Batches 2 and 3
Figure A.5 – Stress-Strain Curve for Samples of #7 Bar

Figure A.6 – Stress-Strain Curve for Samples of #6 Bar
Figure A.7 – Defining Yield Strength of #6 Bars Using 0.2%-Offset Method
Figure A.8 – Front View of Loading Frame and Test Specimen (Dimensions in inches)
Figure A.9 – Dimension Locations for Table A.4 and Table A.5

Figure A.10 – Details of Rectangular Hoops Used in All Beams (Dimensions in inches)
Figure A.11 – Typical Closed Hoop (Dimensions in inches)
Figure A.12 – Details of Wood Formwork (Dimensions in inches)
Figure A.13 – Assembled Wood Formwork
Figure A.14 – Assembled Steel Cages
Figure A.15 – Specimen CC4-X Ready to be Cast
Figure A.16 – Placing and Vibrating Concrete in Formwork
Figure A.17 – Casting 4x8" Cylinders and 4x4x14" Beams
Figure A.18 – Specimens Covered with Wet Burlap and Nylon Sheets After Casting
Figure A.19 – Formwork Removed One Week After Casting
Figure A.20 – Compressive Test of a Sample 4x8” Cylinder
Figure A.21 – Loading Frame, Bottom Beam Details (Dimensions in inches)
Figure A.22 – Loading Frame, Top Beam Details (Dimensions in inches)
Figure A.23 – Loading Frame, Column Details (Dimensions in inches)
Figure A.24 – Top Flange Connection Plates to Bottom Beam (Dimensions in inches)
Figure A.25 – Details of Bottom Support of Specimens (Dimensions in inches)
Figure A.26 – Roller Support Plates (Dimensions in inches)
Figure A.27 – Details of Top Support of Specimens (Dimensions in inches)
Figure A.28 – Bottom Stub Plate (Dimensions in inches)
Figure A.29 – Top Stub Plate (Dimensions in inches)
See Figure A.31 for more details

See Figure A.32 for more details

Figure A.30 – Top Stub Plate to Load Cell Adaptor (Dimensions in inches)
Figure A.31 – Loading Adaptor, Top Part (Dimensions in inches)
Figure A.32 – Loading Adaptor, Bottom Part (Dimensions in inches)
Adjusted South Drift = \((\Delta + w\beta)/a + \beta\)

Adjusted North Drift = \((\Delta - w\beta)/a - \beta\)

Figure A.33 – Calculation of Drift
Figure A.34 – Calibration of the Custom-Made Load Cell against MTS Load Cell

Figure A.35 – Whittemore Points Glued to Surface of Concrete Specimen
Figure A.36 – Electronic Whittemore Gage
LIST OF REFERENCES


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