CYCLIC RESPONSE OF CONCRETE COLUMNS
REINFORCED WITH SAS 670 GRADE-97 STEEL BARS

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Abstract

Since the implementation of Strength Design for structural concrete in the 1960s, US building codes have limited the use of ultrahigh-strength steel (yield strength greater than 80 ksi) for seismic applications. This has been mostly due to insufficient experimental data. The industry has been dominated by the design of concrete members reinforced with steel bars having specified yield strength of 60 ksi (Grade 60). In this study, a series of experiments was designed to investigate the cyclic response of concrete columns reinforced longitudinally with ultrahigh-strength steel bars.

The experiments included two specimens, reinforced with Grade-97 steel bars. A third specimen (tested by others) represented the control specimen reinforced with conventional Grade-60 bars. All three specimens were designed to have similar flexural strengths (with nearly identical \( \rho_s f_y \)) and were subjected to the same loading protocol consisting of reversed-cyclic displacements of increasing amplitude. The applied axial load remained constant throughout the loading protocol. The test specimens included variation in the volume fraction of hooked steel fibers, 0% for reinforced concrete (RC) and 1.5% for high-performance fiber reinforced concrete (HPFRC); and the spacing of the transverse hoops, \( d/4 \) for RC and \( d/2 \) for HPFRC. The nominal concrete strength was 6000 psi for all specimens.

All of the tested columns completed the cyclic loading protocol, which targeted a maximum drift ratio of 5%. The test data indicate that, for RC and HPFRC columns, replacing conventional Grade-60 longitudinal reinforcement with reduced amounts of Grade-97 steel bars attained the target flexural strength and deformation capacity. The calculated nominal and probable moments, using conventional assumptions in Strength Design, were in reasonable agreement with the measured values.
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Chapter 1. Introduction

1.1 Problem Statement

Over the years, reinforced concrete has become a staple of building construction in the United States. As a result, limits and rules were established to ensure a minimum level of safety and quality control for all concrete construction. The introduction of Strength Design, in the body of the ACI building code (ACI 318, 1963), established a 60 ksi limit on the yield strength of all reinforcing steel used in reinforced concrete members. Grade-75 reinforcement was allowed in flexural design provided that full-scale beam testing proved that the average crack width at service load would not exceed 0.015 in. In 1971, the maximum yield strength allowed in reinforcement for flexural and axial loads was increased to 80 ksi. The 60 ksi limit on yield strength was maintained for shear and torsional reinforcement, as well as for any member designed for earthquake-induced forces (ACI 318, 1971). Starting in 2008, ACI 318 allowed the use of confining reinforcement with yield strength of up to 100 ksi, promoting the use of high-strength concrete in construction. Though the transverse reinforcement can now be Gr. 100, concrete members must still be designed for shear using the limit of 60 ksi. All reinforced concrete members resisting seismic-induced forces must also use longitudinal reinforcement with specified yield strength not exceeding 60 ksi. The aforementioned limits were put into place to allow yielding of the longitudinal reinforcement before concrete reaches its ultimate compressive strain, as well as to limit crack widths in the members at service loads. These limits continue to pose challenges for concrete construction in areas where seismic forces govern the design and require excessive amounts of reinforcement.

If the code limits on reinforcement yield strength were relaxed, congestion of reinforcement could be dramatically reduced. A higher limit on yield strength would allow for less steel and potentially smaller sections to attain similar strengths in reinforced concrete members, which would lead to lower material, shipping, and labor costs. Ultimately, using reinforcing steel with high yield strengths would save owners and contractors both time and money while increasing construction quality. Unfortunately, there is insufficient experimental evidence available in the United States to demonstrate
the applicability of higher strength steels as reinforcement in concrete members for earthquake-resistant construction. In this thesis, “high-strength” or “ultrahigh-strength” steel refers to steel having a yield strength in excess of 80 ksi.

In the past five years, there have been three research projects focusing on the use of high-strength steel bars in reinforced concrete frame members for seismic applications. Tavallali (2011) and Pfund (2012), of Penn State University, tested concrete beams reinforced with steel bars having nominal yield strengths of 60, 97, and 120 ksi. Concurrently, Rautenberg (2011) of Purdue University tested concrete columns reinforced with Grade 60, 80, and 120 ksi steel. These tests followed the same loading protocol consisting of a sequence of displacement reversals of increasing amplitudes. The test data showed comparable deformation capacities between the members reinforced with high-strength steel and those reinforced with conventional (Grade 60) reinforcement.

The stress-strain curve for the Gr. 120 steel used in Rautenberg’s column tests has several key differences from the stress-strain curve for conventional Gr. 60 steel—the absence of a sharp yield point, absence of a post-yield plateau, and reduction in elongation at fracture. These differences did not impair the deformation capacity of the columns in relation to the conventionally-reinforced columns. To further understand the behavior of concrete columns reinforced with ultrahigh-strength steel bars, experimental data are needed for cases involving high-strength reinforcement with a sharp yield point and without appreciable strain hardening (tensile strength-to-yield strength ratio of less than 1.2). The research proposed in this study will address this void.

The experiments by Rautenberg (2011), Tavallali (2011), and Pfund (2012) included test specimens with high-performance fiber reinforced concrete (HPFRC), defined here as a class of fiber reinforced concrete that shows strain-hardening behavior after first cracking (Parra-Montesinos, 2005). Because HPFRC increases the usable compressive and tensile strains of concrete, its combined use with ultrahigh-strength reinforcement is likely to allow the effective use of yield strength in excess of 80 ksi with enhanced control of concrete spalling and cracking. The test specimens with HPFRC exhibited improvements in flexural and shear strengths, in relation to the reinforced concrete (RC) specimens without fibers.
1.2 Objectives and Scope

The objective of this research program was to study the cyclic response and deformation capacity of columns reinforced longitudinally with high-strength steel characterized by a sharp yield point and without appreciable strain hardening. The objective was pursued through testing two specimens with similar flexural strength under transverse displacement reversals and comparing to a control specimen tested by Rautenberg (2011) with similar flexural strength, dimensions, test setup, and loading protocol. The product of the longitudinal reinforcement ratio and the yield strength of the reinforcement was nearly identical for all specimens. The study of specimen behavior focused on force vs. displacement response, strain measurements at selected points along the longitudinal and transverse reinforcement, crack widths along the member length, and stiffness reductions.

The experimental program for this study considered three variables:

The volume fraction of steel fibers in the concrete matrix, \( V_f = 0\% \) or 1.5\% (i.e., RC or HPFRC), and the spacing of transverse reinforcement, \( s = d/4 \) in RC or \( d/2 \) in HPFRC.

An additional variable exists between the specimens tested for this study, and the control specimen referenced from Rautenberg (2011)—The yield strength of the longitudinal reinforcement, \( f_y = 60 \) ksi (Rautenberg, 2011) or \( f_y = 97 \) ksi (this study).

Five parameters were kept constant in the specimens tested for this study, as well as in the control specimen. The nominal values for these parameters were:

- Cross-sectional dimensions, \( b = 9 \) in. and \( h = 12 \) in.;
- Concrete compressive strength, \( f'_c = 6000 \) psi;
- Yield strength of the transverse reinforcement, \( f_{yt} = 60 \) ksi;
- Shear span-to-effective depth ratio, \( a/d = 3 \);
- Applied axial load, \( 0.2 f'_c A_g \).
1.3 Previous Research

This section provides a brief review of U.S. research concerning high-strength steel and fiber reinforced concrete for seismic applications.

1.3.1 High-Strength Reinforcement

The limits on specified yield strength of reinforcement in ACI 318-63 (1963) and ACI 318-71 (1971) were the result of the research conducted by Richart and Brown (1934), Hognestad (1961), and Todeschini et al. (1964), among others. None of these tests addressed the behavior of reinforced concrete members subjected to reversed-cyclic transverse loading. Richart and Brown (1934) report the test results of 564 circular columns, concentrically loaded, 16 of which were reinforced longitudinally with steel bars having yield strength of 80 to 96 ksi. The report indicates that, while high-strength steel is effective in carrying load, the working stresses may be limited by the permissible column deformations (shortening).

Hognestad (1961) and collaborators participated in eight reports from 1961 to 1966, discussing the results from experimental research conducted at the PCA laboratories to address performance requirements for concrete members reinforced with high strength steel (Hognestad, 1961, 1962; Gaston and Hognestad, 1962; Kaar and Mattock, 1963; Pfister and Mattock, 1963; Pfister and Hognestad, 1964; Kaar and Hognestad, 1965; Kaar, 1966). Hognestad (1962) addressed the topic of crack behavior and crack control, finding that bar size, reinforcement ratio, concrete compressive strength, beam width, and beam depth played a secondary role on crack width. The concrete clear cover to the flexural reinforcement was found to be the most influential parameter on crack width. Hognestad also found that all major cracks manifested by the time the steel stress had reached 30 ksi and that concrete crack widths were proportional to the stress developed in the reinforcing bars.

Todeschini et al. (1964) built upon Richart and Brown (1934) and Hognestad (1961) with the testing of 19 11-in. square columns, concentrically and eccentrically loaded, reinforced longitudinally with steel bars having yield strength in excess of 75 ksi. Transverse reinforcement consisted of rectangular ties at 10-in. spacing. Todeschini et al.
(1964) found that the columns failed as a result of concrete crushing before the longitudinal reinforcement buckled, with the concrete most commonly experiencing compressive strains of 0.003 or higher.

Restrepo et al. (2006) conducted cyclic testing of two 35%-scale bridge-column units, one fully reinforced with high-strength steel (yield strengths of 94 ksi for longitudinal reinforcement and 135 ksi for transverse reinforcement), the other with all Grade-60 reinforcing steel. Restrepo et al. found that both bridge specimens could sustain drift ratios of 3.9% without failure, the high-strength steel specimen had smaller initial stiffness and larger yield deflections.

ACI ITG-6R-10 (2010) presents a description of material characteristics, design criteria, and example calculations for the use of ASTM A1035 Grade 100 steel bars in concrete beams, columns, slabs, walls, and footings. NCHRP Report 679 (Shahrooz et al., 2011) reported on the use of ASTM A1035 reinforcing steel and recommended specification language to allow the use of high-strength reinforcing steel with specified yield strength not greater than 100 ksi. Both of these documents, ACI ITG-6R-10 and NCHRP Report 679, did not focus on seismic applications of concrete members reinforced with high-strength steel.

Tavallali (2011) tested reinforced concrete (RC) beams with high-strength steel bars (Grade 97, SAS 670 threaded bars) used as primary flexural reinforcement. His experimental program also explored the combined effects of high-performance fiber reinforced concrete (HPFRC) and Grade-97 flexural reinforcement. Control specimens, in RC and HPFRC, were reinforced with conventional Grade-60 steel bars. HPFRC specimens explored the use of one-half the amount of transverse reinforcement that was used in RC beams. The drift ratio history applied to the specimens followed the protocol of FEMA 461 (2007) up to 5% drift ratios with the addition of a monotonic final push. The RC beam tests showed that replacing Grade-60 reinforcement with reduced amounts of Grade-97 steel bars (reduced in proportion to the increased yield strength) led to nearly identical flexural strength and did not decrease the usable deformation capacity. The reduced amounts of transverse reinforcement in the HPFRC beams did not reduce the strength or deformation capacity. The tested RC and HPFRC beams tolerated the cyclic loading protocol to 5% drift ratios without failure. However, at 60% of the peak load, maximum
crack widths in RC beams reinforced with Grade-97 steel bars were approximately 1.7 times larger than the maximum crack widths measured in RC beams reinforced with Grade-60 bars. The maximum crack widths in HPFRC beams reinforced with Grade-97 steel bars were generally smaller than in RC beams reinforced with Grade-60 bars. First-yield of longitudinal reinforcement for the RC beams reinforced with Grade-97 steel bars occurred at drift ratios approximately 1.5 times greater than that for RC beams reinforced with Grade-60 bars, which led to a reduction in the area enclosed by the measured force-displacement hysteresis loops for the RC beams with Grade-97 bars.

Rautenberg (2011) tested reinforced concrete (RC) columns with steel bars having yield stress of 60, 80, or 120 ksi. One specimen used high-performance fiber reinforced concrete (HPFRC) with increased hoop spacing, similar to the research program of Tavallali (2011). The specimens were also subjected to a cyclic loading protocol patterned after FEMA 461 (2007). Axial load was kept constant throughout the test at either 0.1 or 0.2$f_c^c A_g$. The tests showed that the columns longitudinally reinforced with Grade-80 and Grade-120 steel bars had comparable flexural strengths and deformation capacities to the columns reinforced with Grade-60 bars. RC columns with transverse reinforcement spaced at $d/4$ had drift capacities between 4 and 8%. The RC columns (reinforced with either Grade-60 or Grade-120 steel bars) subjected to axial loads of $0.2f_c^c A_g$ failed after completing the first cycle of 5% drift ratio, where the mode of failure was buckling of the longitudinal reinforcement. The HPFRC column (reinforced with Grade-120 steel bars) subjected to an axial load of $0.2f_c^c A_g$ completed the two cycles of 5% drift ratio but failed during the final push, where the mode of failure was fracture of the longitudinal reinforcement.

As an extension to the research program of Tavallali (2011) and Rautenberg (2011), Pfund (2012) tested four beam specimens, two of which were RC and two HPFRC. The two RC specimens included a control specimen reinforced with Grade-60 steel bars (ASTM A706 steel) and a specimen with Grade-120 bars (ASTM A1035 steel). The HPFRC specimens included a specimen reinforced with Grade-120 bars (ASTM A1035 steel) and a specimen with Grade-97 bars (SAS 670 threaded bars). All transverse reinforcement were hoops of Grade-60 steel. All beams completed the cyclic loading protocol, also following FEMA 461 (2007), with a target maximum drift ratio of 5%. The test data showed that replacing conventional Grade-60 flexural reinforcement with reduced amounts of Grade-
97 or Grade-120 steel bars (reduced in proportion to the increased yield strength) led to similar flexural strength and deformation capacity. None of the types of reinforcement used, with very different stress-strain curves, had a negative effect for completing the loading protocol to 5% drift.

Many research projects have been conducted regarding the use of high strength steel as transverse reinforcement as well. Wight and Sozen (1973) evaluated 12 reinforced concrete columns and found that the rate of strength decay, in columns subjected to large displacement reversals, was a function of axial load, percentage of transverse reinforcement, and transverse deflection. They found that strength decay was generally triggered by the yielding of transverse reinforcement. Their findings suggest that the use of high-strength steel as transverse reinforcement would help mitigate strength decay. Muguruma et al. (1991) found that high-strength steel used as transverse reinforcement improves flexural ductility and delays longitudinal bar buckling. Sato et al. (1993) also discussed the use of high strength-steel as transverse reinforcement to prevent buckling of the longitudinal reinforcement. Lin and Lee (2001) found that the use of high-strength transverse reinforcement with yield strengths greater than 60 ksi increase the ductility and can reduce the amount of confining reinforcement required to achieve the same ductility. Pessiki et al. (2001) developed a procedure for analysis and design of high-strength spiral reinforcement. Budek et al. (2002) reported findings from nine columns transversely reinforced with high-strength steel, also confirming the benefit of delaying bar buckling, as well as providing recommendations for the effective use of high-strength steel as transverse reinforcement.

The present study does not address the effects of high-strength steel used as transverse reinforcement. The test specimens that are part of this study had transverse reinforcement with a specified yield strength of 60 ksi.

1.3.2 Fiber Reinforced Concrete

Fiber-reinforced concrete (FRC), as defined by ACI 116R, is “concrete containing dispersed randomly oriented fibers”. Summaries of the research, history, and applications of FRC have been presented by Zollo (1997), Li (2002), and Parra-Montesinos (2005). Zollo (1997) covers 30 years of FRC since the 1960s, describing the background, technology,
behavior, and applications of the FRC matrix. Li (2002) approaches the history of FRC from a Civil Engineering standpoint, discussing the necessary characteristics for industry success, current (mainly nonstructural) applications, as well as potential future structural applications. Parra-Montesinos (2005) provides a summary of research projects that have focused on seismic applications of FRC.

Research on FRC began with Romualdi and Batson (1963) and Romualdi and Mandel (1964). Romualdi and Batson tested oriented wire-reinforced concrete, while Romualdi and Mandel progressed to testing shorter, dispersed, and randomly oriented wires in the concrete matrix. The key finding in Romualdi and Mandel was the observation that the concrete matrix does not immediately rupture at the cracking stress but, with an appropriate content of short wires, exhibits substantial post-cracking strength. Swamy and Al Noori (1975) proposed that FRC could be used to reduce the size of cracks and promote the use of high-strength steel reinforcement in concrete members.

Fanella and Naaman (1985) studied the effects of fibers made of steel, glass, and polypropylene on the concrete matrix in compression. They found that fibers mainly influence the post-peak descending branch of the stress-strain curve of concrete, leading to higher ductility and toughness. Craig (1987) tested 13 FRC beam specimens to investigate matrix behavior with different variables, and found that fibers increased stiffness, flexural strength, ductility, and ultimate concrete compressive strain. Spalling and bar buckling were prevented and cracks were observed to be more uniformly dispersed.

Soroushian and Bayasi (1987, 1991) studied the effects of fiber type, aspect ratio, and volume fraction on matrix behavior, concluding that randomly distributed fibers improve concrete toughness, impact resistance, and tensile strength. They stated that hooked steel fibers were more effective than straight ones at improving flexural and compressive behavior in concrete. Hota and Naaman (1997) found that a higher volume fraction of fibers resulted in higher bond strength between reinforcing bars and concrete.

As stated above, Li (2002) presented an overview of fiber applications in cementitious composites for the civil engineering industry, observing that most applications are nonstructural. Recently, more effort has been devoted to the use of fibers for solving structural problems with high-performance fiber reinforced concrete (HPFRC).
members. As defined in Section 1.1, 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. To achieve strain-hardening and multiple cracking in tension, a minimum volume fraction of fibers (referred to as the critical volume fraction) should be used so that the maximum post-cracking stress is greater than or equal to the stress at first cracking.

Over the last ten years, several researchers have studied the behavior of HPFRC members for seismic applications. Fischer and Li (2002) studied the effect of matrix ductility on the cyclic response of HPFRC flexural members with conventional Grade-60 steel bars. They investigated using HPFRC as a substitute to transverse reinforcement. Their test data showed that the shear resistance provided by stirrups for confinement was redundant in the HPFRC specimens. HPFRC members were found to have a higher damage threshold than RC members. The specimens tested by Fischer and Li were subjected to shear stresses in the order of $2.5\sqrt{f_c}$ [psi], too low for extrapolating their findings to the more severe stresses commonly found in practice.

Parra-Montesinos et al. (2005) explored the use of HPFRC for eliminating 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%. They concluded that the ACI 318 code limitations on joint shear stresses (i.e., $20\sqrt{f_c}$ [psi] for a joint confined on all four sides) could be safely applied to HPFRC beam-column connections without transverse reinforcement. Later, Parra-Montesinos and Chompreda (2007) found that for seismic applications of HPFRC beams, a shear stress of $3.5\sqrt{f_c}$ [psi] is safe for determining the concrete contributions to shear strength ($V_c$).

The cyclic tests of Tavallali (2011) and Pfund (2012) included beam specimens with HPFRC. They found that HPFRC beams (1.5% volume fraction of hooked steel fibers) were approximately 20% stronger in flexure than the RC equivalent members. Tavallali and Pfund also found that the concrete contribution to shear resistance could be as high as $V_c = 4\sqrt{f_c}$ [psi], and that HPFRC specimens with high-strength steel generally exhibited smaller crack widths than the RC specimens with Grade-60 bars. Fibers also were observed to reduce the rate of stiffness degradation with increased cyclic deformations.
The experimental program of Rautenberg (2011) included one HPFRC column specimen reinforced with Grade-120 steel bars and subjected to large displacement reversals. The HPFRC specimen was the only column, out of the six tested under an axial load of $0.2f'_c A_g$, that completed the loading protocol (per FEMA 461, 2007) to drift ratios of 5%.
Chapter 2. Experimental Program

2.1 Introduction

Two specimens were tested under repeated and increasing load reversals to explore the deformation capacity of concrete columns reinforced with high-strength steel as primary flexural reinforcement. This chapter provides a summary of the experimental program for these tests.

2.2 Specimen Description

Each of the two test specimens consisted of a central stub that functioned as the base of a cantilever for two columns extending from opposite sides of the stub. These columns were supported by rollers 30 in. past the face of the central stub on each side. Each specimen was 88-in. long, from end to end, with a central stub that was 16-in. long, and columns that were 36-in. long. Figure 2.1 shows the nominal dimensions for both specimens, including roller support locations. Cross-sectional dimensions of the columns and stub were the same for both specimens. The column cross-section was 9-in. wide by 12-in. deep, and the stub cross-section was 9-in. wide by 22-in. deep. The dimensions of the column cross-section were chosen to attain a maximum shear stress \(\frac{V}{bd}\) of approximately \(7\sqrt{f'c}[\text{psi}]\) for an actuator load of 100 k applied via the central stub. Table 2.1 shows a summary description of the specimens. Section 3 of Appendix A contains further information on the specimens, including as-built dimensions.

Reinforcement details for the specimens were chosen so that the specimens had similar flexural strengths to the control specimen from Rautenberg (2011), CC-2.4-20, described in Appendix B. Figure 2.2 provides a schematic view of the reinforcement layout for the two tested specimens. Longitudinal reinforcement for both specimens consisted of four Grade 97 (SAS 670) longitudinal bars with a nominal diameter of 0.71 in. (18 mm) and a cross-sectional area of 0.39 in.\(^2\) (250 mm\(^2\)), approximating an American #6 reinforcing bar. Transverse reinforcement in the form of closed hoops complies with the requirements for special moment frame columns as specified in Chapter 21 of ACI 318-08 (2008). The
hoops confine the concrete core, help mitigate longitudinal reinforcing bar buckling, resist shear stresses, and enhance the bond of the concrete to the flexural reinforcement. Transverse reinforcement in all specimens was made of ASTM A615 Grade 60 deformed #3 bars. For the high-performance fiber reinforced concrete (HPFRC) specimen, Dramix RC-80/30-BP hooked steel fibers were mixed into the concrete. These fibers had a tensile strength of 330 ksi (2300 MPa), length of 1.2 in. and diameter of 0.015 in., for an aspect ratio of 80. Additional details on the material properties can be found in Appendix A.

There were three experimental variables in the specimens—the yield strength of the longitudinal reinforcement (60 or 97 ksi); the volume fraction of fibers (0% for the RC specimen, or 1.5% for the HPFRC specimen), and the spacing of the transverse reinforcement (d/4 for the RC specimen, or d/2 for the HPFRC specimen). The controlled parameters for the specimens included the cross-sectional dimensions of the columns, b and h; nominal compressive strength of the concrete, f’c; nominal yield strength of the transverse reinforcement, f’y_t; the shear span-to-effective depth ratio, a/d; and the applied axial load, P:

\[
b = 9 \text{ in. and } h = 12 \text{ in.}; \\
f'c = 6000 \text{ psi}; \\
f_{y_t} = 60 \text{ ksi} \\
a/d = 3.0 \\
P = 0.2 f'c A_g
\]

The specimen naming convention (see Table 2.1) involved the use of five different properties, represented by two characters, two numbers, and then a final optional character. The first character, “U”, refers to the use of ultrahigh-strength steel as longitudinal reinforcement, indicating a nominal f_y of 97 ksi. This can be compared to the control specimen discussed in Appendix B, which had a first character of “C” for conventional strength longitudinal reinforcement with a nominal f_y of 60 ksi. The second character, “C”, stood for conventional strength transverse reinforcement with a nominal f_y of 60 ksi. The first number, “1.5”, refers to the longitudinal reinforcing ratio (based on gross section), 1.5%, as compared to the control specimen, which had a first number of “2.4” for a 2.4% longitudinal reinforcement ratio. The second number, “20”, refers to the amount of axial
load applied, for 20% of $f'cA_g$. The final optional character at the end of the specimen name was “F”, which indicates the presence of hooked steel fibers in the concrete matrix.

2.3 Test Procedure

Before administration of the loading protocol (Table 2.2), the axial load was applied to the specimens using an external post-tensioning system. The axial load was monitored and adjusted continuously throughout the test procedure to remain within 3% of the target axial load. Additional information on the axial load can be found in appendix A, section 4.1.

The loading protocol was administered by applying transverse (lateral) load through the central stub of the specimen. Both specimens were subjected to the same 12-step loading protocol, where each step had a target drift ratio that was approximately 40% larger than the previous step, which took the specimens from 0.15% drift ratio to 5% drift ratio. In this document, drift ratio is defined as the ratio of transverse displacement to shear span. Each step consisted of two full cycles, pushing each specimen first to the positive drift ratio peak, then pulling to the negative drift ratio peak, then again to the positive peak, followed by the negative peak, and then to zero transverse load. This loading protocol, shown in Table 2.2 and Figure 2.3, follows the recommendations in FEMA 461 for the quasi-static cyclic testing of structural and nonstructural components. The loading protocol was completed at the end of the second cycle at a drift ratio of 5%. Once the specimens were returned to zero transverse load at the end of step 12, the specimens were subjected to a monotonic push to failure, or until the specimen reached the maximum drift amplitude attainable by the test setup. A typical test setup is shown in Figure 2.4.

The individual columns of each specimen shared similar drift ratios during the early cycles of the loading protocol, typically below 1% drift ratio. At increasing displacements, the columns experienced differing drift ratios due to yielding of the longitudinal reinforcement on one side of the stub, which induced rotation of the stub. Typically, after step 8 of the loading protocol, one of the two columns experienced a consistently greater drift ratio than the other, leading to that column controlling the loading protocol. All
reported drift ratios, for each of the two columns, account for the stub rotation. Section 4 of Appendix A provides a detailed description of the test procedure.

### 2.4 Instrumentation

The specimens were instrumented so that the lateral load, axial load, drift ratio, and reinforcement strains were continuously recorded during the test. Concrete surface strains were recorded at the second positive peak, second negative peak, and final zero transverse load for each step.

Transverse load and axial load were both measured using a series of load cells. The transverse load was measured using a load cell consisting of three strain gages, wired together and attached to the plunger of the hydraulic cylinder that was used to administer the loading protocol, as seen in Figure 2.5. Axial load was measured using custom strain gage circuitry that was attached to each of the four post-tensioning bars used for application of the axial load.

The drift ratios were monitored via displacement sensors. Eight linear potentiometers were positioned according to the schematic view in Figure 2.6, and measured the change in displacement of the specimen during the test. The displacement measurements were used to continuously correct for rigid body movements in the specimen and then to calculate, report, and record the drift ratio for each column throughout the test. All of the calculations were automated using the software from the data acquisition system.

Reinforcement strains on the longitudinal and transverse reinforcement were also recorded and reported continuously throughout the test. The strain gages were located at the positions shown in Figure 2.7.

Concrete surface strains were measured using electronic Whittemore gages that registered the deformations between steel buttons adhered to the face of each specimen in an 8-in. by 8-in. grid, as shown in Figure 2.8. The Whittemore gage instruments (Figure A.21) were designed by Pfund (2012) for variable grid dimensions.
Crack widths were also measured at the peaks of all cycles, but were measured and recorded manually, requiring no additional instrumentation. Greater detail about the instrumentation and measurements is provided in section 4.2 of Appendix A.
Chapter 3. Experimental Results

3.1 Introduction

This chapter presents the experimental data obtained from the specimens built and tested as part of this study. The data include applied shear, displacements, strains in the longitudinal reinforcement, strains in the transverse reinforcement, crack widths, and curvatures. The reported drift ratios were continuously calculated using the readings from the displacement sensors, which are described in Chapter 2 and Appendix A. The reported shear values were calculated by dividing the force applied by the hydraulic cylinder, attached to the central stub, by two. Photographs of the specimens were taken at each peak, as well as at zero shear conditions, to document the damage state throughout the tests. Black marker lines were drawn alongside cracks to increase their visibility in photographs. Additional descriptions for strain gage locations and overall instrumentation can be found in Appendix A.

3.2 General Observations

Specimen descriptions are presented in Table 2.1. A summary of the test results, including maximum drift, shear, and shear stresses for both specimens, is shown in Table 3.1. Both specimens successfully completed the loading protocol, after which, they were pushed monotonically to drift ratios in excess of 10%. Spacing of the transverse reinforcement for the RC specimen (UC-1.5-20) followed the seismic design provisions in ACI 318 (2008) for columns in special moment frames, with the maximum spacing limited to \( s = d/4 \), where \( d \) is the effective depth of the cross section. For the HPFRC specimen (UC-1.5-20F), spacing of the transverse reinforcement was increased to \( s = d/2 \) to investigate the effect of hooked steel fibers on shear resistance and confinement.

UC-1.5-20 was designed with SAS 670 Grade 97 longitudinal reinforcing bars with a diameter of 0.71 in. (18 mm), about the size of an American #6 bar. Transverse reinforcement consisted of conventional Grade 60 #3 hoops with 135° seismic hooks. This specimen completed the loading protocol successfully. During the test, there was a
displacement sensor malfunction while reaching the first positive peak of step 11 (4\% drift ratio), leading to a deviation from the loading protocol during step 11, where the negative peaks reached 5\% drift ratio and the second positive peak reached 3\% drift ratio (instead of the target 4\% drift ratio). The sensor was repaired and the specimen completed the rest of the loading protocol. The final monotonic push brought the controlling south column to a deformation of 10\% drift ratio. The final push was halted when the sensors reached test apparatus limits.

Photos of specimen UC-1.5-20 at various drift ratios are presented in Figure 3.1. This specimen experienced only flexural cracking at drift ratios below 0.4\% (steps 1 to 4 of the loading protocol). The first inclined cracks were observed in the first cycle of step 5 (0.6\% drift ratio). The widest flexural cracks occurred near the face of the stub on the south column, which controlled the loading protocol. Flexural cracks were too wide (>0.06 in.) to be measured with a crack comparator on the south column in the first cycle of step 10 (3\% drift ratio). Shear cracks on the south column reached a size of 0.04 in. in the first cycle of step 10 (3\% drift ratio), but did not get noticeably wider in the following cycles. Figure 3.3 shows a photograph of specimen UC-1.5-20 at the end of the test, with major concrete spalling in the controlling south column. Minimal spalling occurred until the south column reached 4\% drift ratio (step 11). Damage to the specimen was concentrated at the interface of the stub with the south column.

Similar to specimen UC-1.5-20, specimen UC-1.5-20F was designed with the same SAS 670 Grade 97 longitudinal reinforcing bars, Grade 60 transverse hoops spaced at \(d/2\), and a volume fraction of hooked steel fibers \(V_f = 1.5\%.\) UC-1.5-20F successfully completed the loading protocol controlled by the south column. The final monotonic push brought the specimen to drift ratios in excess of 10\% without failure. The final push was halted when the sensors reached test apparatus limits.

Figure 3.2 shows photographs of UC-1.5-20F at various drift ratios. This specimen experienced only flexural cracking at drift ratios of 0.4\% or less (steps 1 to 4 of the loading protocol). The first inclined cracks were observed in the first cycle of step 5 (0.6\% drift ratio). A primary flexural crack that was too wide (>0.06 in.) to be measured by the crack comparator developed at the face of the stub on the south column after fiber pullout at 3\% drift ratio (step 10). The controlling south column experienced maximum shear crack
widths in excess of 0.016 in. during the first cycle of step 10 (3% drift ratio). The largest flexural and shear cracks occurred around the stub face. A photograph of specimen UC-1.5-20F at the end of the test can be found in Figure 3.4. At drift ratios of 10%, spalling was very limited and concentrated around the fiber pullout region near the interface of the central stub with the south column.

3.3 Measured Response

3.3.1 Shear vs. Drift Ratio Curves

The shear vs. drift relationships for both columns of both specimens are shown in Figures 3.5 to 3.12. Figures 3.5 to 3.8, show the hysteresis loops of each column during the cyclic loading protocol, Figures 3.9 and 3.10 correspond to the final monotonic push of the controlling columns for both specimens, and Figures 3.11 and 3.12 show the envelopes of the shear vs. drift ratio curves.

In this study, drift is defined as the transverse deflection of the column between the points of zero and maximum moments. Drift ratio is the drift divided by the shear span, which is 30 in. for both columns of both specimens. All reported drift ratios have been adjusted to account for stub rotation. Details on calculating the drift ratio can be found in Appendix A, section 4.2.

For a single specimen, both columns performed similarly during the early cycles of the test, for which the average drift ratio of both columns was used to control the target drift ratio in the loading protocol (Table 2.2). Later, in the cycles at or above 1% drift ratio, one column of the specimen drifted further than the other. That column, which was the south side for both specimens, then controlled the loading protocol. Once the loading protocol was completed, each specimen was subjected to monotonic loading (Figures 3.9 and 3.10) until the specimen either failed, or the limits of the test apparatus were reached. The envelope curves for the controlling columns were determined using the peak shear and peak drift ratio from each step. A step is defined as two full cycles at a single target drift ratio (refer to Table 2.2 and Figure 2.3). Maximum shear for both specimens is presented in Table 3.1. Comparisons of the shear vs. drift ratio relationships for both
specimens (UC-1.5-20 and UC-1.5-20F) and the control specimen (CC-2.4-20) are reported in Chapter 4.

3.3.2 Strains in Longitudinal Reinforcement

Strain in the longitudinal bars was recorded continuously during the tests using strain gages at selected locations, identified in Appendix A. Longitudinal strains were measured and reported for the duration that the strain gages remained functional, which was generally below 60,000 microstrain (6% elongation).

Figures 3.13 and 3.14 show the measured strain vs. drift ratio and strain vs. shear relationships for specimen UC-1.5-20. The strain plots show that, at cycles beyond 3% drift ratio, the longitudinal bars experienced strains exceeding yield at all the points where strain was measured (see Figure 2.7), an indication that the plastic hinge length was at least 10 in. (i.e., the effective depth of the column, \(d\)). At the south column-stub interface, the bottom bars experienced higher strains than the top bars by approximately 70%, which could be explained by the order in which the bars yielded. As noted in Table 3.2, the first location (at the face of the stub) to experience yield strains was the bottom bars on the south column at a drift ratio of 0.8%. The bottom bars on the north column showed yield strains next, at 1.38% drift ratio, followed by the top bars on the north column at -0.93% drift ratio, then the top bars on the south column at -1.45% drift ratio.

Figures 3.15 and 3.16 show the measured strain vs. drift ratio and strain vs. shear relationships for specimen UC-1.5-20F. The strain gage readings on the controlling south column during cycles at 3% drift ratio exceeded the yield strain, indicating that the longitudinal bars yielded even at a distance \(d\) (effective depth) away from the face of the stub. The top bars of the south column-stub interface experienced larger strains than the bottom bars in the same location, though not as significant of a difference in strains as observed in UC-1.5-20. The top bars on the south column exceeded yield strains at an average drift ratio of -0.87%, then the bottom bars on the north column at 1.14% drift ratio, followed by the south column bottom bars at 1.20% drift ratio, then finally the north column top bars at -1.23% drift ratio.
3.3.3 Strains in Transverse Reinforcement

For each specimen, two hoops of each column were selected for strain measurements. For UC-1.5-20, the first and third hoops from the face of the stub were instrumented with strain gages, while for UC-1.5-20F, the first and second hoops were the ones instrumented. The instrumented hoops for both specimens were located at approximately 1.25 in. and 6.25 in. away from either face of the stub. Details of the strain gage setup are detailed in Appendix A (strain gage locations are shown in Figure 2.7).

Figures 3.17 and 3.18 show the measured strain vs. drift ratio and strain vs. shear relationships for the hoops of specimen UC-1.5-20. A comparison of the plots shows that the hoop closer to the south column-stub interface experienced lower strains than the hoop that was 6.25 in. away from the south column-stub interface for most of the test, until the inner hoop reached strains exceeding yield, which occurred during step 12 (5% drift ratio) of the loading protocol. Figure 3.1 and Figure 3.3 show the presence of a large inclined crack crossing the outer hoop (at 6.25 in. from the face of the stub), which is located just outside the spalled area on the south column of the specimen. All the plots show that tensile strain accumulated in the hoops with every cycle, and that the north column hoops never reached strains exceeding yield.

Figures 3.19 and 3.20 show the measured strain vs. drift ratio and strain vs. shear relationships for the hoops of specimen UC-1.5-20F. The plots show that none of the hoops reached yield strains. Similar to UC-1.5-20, the hoops nearest the face of the stub experienced lower strains than the hoops at 6.25 in. from the face of the stub.

3.3.4 Curvatures

Moment vs. Curvature relationships are based on the use of the applied shear multiplied by the shear span to define the moment, and on the longitudinal strain measurements of the top and bottom bars at face of the stub divided by the distance between the top and bottom bars to define curvature. Additionally, the mean curvature distribution along the specimens was determined by using the data collected from Whittemore gage measurements, as shown in Figures 2.8 and 3.21. The mean curvature
was calculated by adding the mean strain at the top and bottom of each station, then dividing by the vertical dimension of the station. The mean strains at each station were found by dividing the horizontal Whittemore gage readings by the distance between gage points measured at the start of the loading protocol.

Figures 3.22 and 3.23 present the moment vs. curvature relationships at the face on either side of the central stub for both specimens UC-1.5-20 and UC-1.5-20F. Figures 3.24 to 3.27 present the mean curvature distributions along the length of each column for both specimens. Each plotted line in Figures 3.26 and 3.27 represents the curvature distribution from the south column roller support to the north column roller support at the peak of the second cycle of a step of the loading protocol. Data is unavailable for specimen UC-1.5-20 due to a malfunction of the Whittemore software that took place during testing. Mean curvature distribution data for specimen UC-1.5-20F was limited to steps with target drift ratios of 1.5% or less because the formation of cracks dislodged a Whittemore contact point from the surface of the specimen. As is apparent from the figures, the curvature is highest just outside the stub face. The topmost line in both plots, representing the curvature distribution at 1.5% drift ratio, is associated with conditions where the longitudinal reinforcement has yielded.

3.3.5 Crack Widths

Limiting the width of cracks is an important part of concrete member design for working stresses, especially when using high-strength steel reinforcement where higher stresses are induced in the steel, resulting in wider cracks that could pose potential problems. Although crack widths are generally not a concern in columns, crack widths are reported here for the sake of completion.

While the current version of ACI 318 does not provide an upper limit to calculated crack width, earlier versions of ACI 318 (before 1999) provided design recommendations based on a calculated maximum crack width of 0.016 in. Figures 3.28 to 3.31 show the maximum measured crack widths at the positive and negative peaks of each cycle for both specimens, with a horizontal line plotted at the 0.016 in. crack width limit used in earlier versions of ACI 318.
For specimen UC-1.5-20, flexural crack widths began exceeding 0.016 in. after the 10th cycle (step 5, 0.60% drift ratio). Shear cracks reached widths of 0.016 in. when the specimen finished the 16th cycle (step 8, 1.5% drift ratio). The addition of fibers to specimen UC-1.5-20F helped reduce the maximum measured crack widths. Flexural cracks exceeded 0.016 in. at larger drift ratios, after the 14th cycle (step 7, 1% drift ratio), while shear cracks reached widths of 0.016 in. during the 17th cycle (step 9, 2% drift ratio).

3.3.6 Shear Deformations

Shear deformations were determined using the Whittemore gage readings. Whittemore measurements were taken at the peaks (positive and negative) of the second cycle at each loading step (refer to Appendix A, Section 4.3). The angular changes in the Whittemore stations were assumed to be the result of bending, shear, and core expansion (Figure 3.32). Due to loss of Whittemore gage contact points as a result of concrete cracking, shear deformation calculations were limited to cycles with peak drift ratios of 2% or less, which correspond to conditions where the transverse reinforcement had not reached yield strains in either specimen UC-1.5-20 or specimen UC-1.5-20F; therefore, core expansion was assumed to be negligible.

The average shear distortion angle at a step n of the test (in relation to initial values, i) was calculated using the interior angles of each Whittemore station, as labeled in Figure 3.32:

\[
\Delta A = A_n - A_i \approx \frac{\theta}{2} + v
\]  
(3.1)

\[
\Delta B = B_n - B_i \approx \frac{\theta}{2} - v
\]  
(3.2)

\[
\Delta C = C_n - C_i \approx -\frac{\theta}{2} + v
\]  
(3.3)

\[
\Delta D = D_n - D_i \approx -\frac{\theta}{2} - v
\]  
(3.4)
And either Equation 3.5 or Equation 3.6, depending on the Whittemore data available for that specimen:

\[ \Delta A - \Delta B + \Delta C - \Delta D = 4\nu \]  
\[(3.5)\]

\[ \Delta A - \Delta B = 2\nu \]  
\[(3.6)\]

A, B, C, and D are the four angles of an individual Whittemore station, calculated using the law of cosines:

\[ A = \cos^{-1}\left( \frac{h_f^2 + v_l^2 - d_2^2}{2h_f v_l} \right) \]  
\[(3.7)\]

\[ B = \cos^{-1}\left( \frac{h_r^2 + v_r^2 - d_1^2}{2h_r v_r} \right) \]  
\[(3.8)\]

\[ C = \cos^{-1}\left( \frac{h_b^2 + v_r^2 - d_2^2}{2h_b v_r} \right) \]  
\[(3.9)\]

\[ D = \cos^{-1}\left( \frac{h_b^2 + v_l^2 - d_1^2}{2h_b v_l} \right) \]  
\[(3.10)\]

As shown in Figure 3.33, it was assumed that shear deformations mainly occurred in the areas instrumented with steel contact points, while the stations labeled as 4 and 4' in the figure were assumed not to contribute to shear deformations. To find the shear contribution to drift of a particular Whittemore station, the average calculated shear distortion for that station was multiplied by the nominal horizontal dimension between station points, 8 in. To find the total shear deformation, the contributions of the three stations located on either side of the specimen centerline were summed, as shown below:
\[ \Delta_{\text{shear}} = (v_1 + v_2 + v_3) \cdot 8 \text{ in.} \]

(3.11)

Finally, to find the shear contribution to the total drift ratio, the total shear deformation was divided by the shear span, \(a = 30\) in. The calculated shear components of drift for the controlling column of both specimens is plotted against total drift in Figures 3.34 and 3.35. The figures show that shear deformations were comparable for both specimens and account for less than 20% of the total deformation.
Chapter 4. Analysis of Measured Response

4.1 Introduction

Chapter 4 compares the response of specimens UC-1.5-20, UC-1.5-20F, and the control specimen, CC-2.4-20 (Rautenberg, 2011). The following sections focus on shear vs. drift ratio history, strains of longitudinal reinforcement, crack widths, stiffness reductions, drift components, and the hysteresis model used for representing the column force-displacement response. The data analyzed in this chapter correspond to the experimental data documented in Chapter 3, Appendix A, and Appendix B.

4.2 Maximum Shear

All three specimens (CC-2.4-20, UC-1.5-20, and UC-1.5-20F) were designed to have similar flexural strength, which resulted in required reinforcement ratios that were inversely proportional to the yield strength of the longitudinal reinforcement. The maximum applied shear for all three specimens is plotted as a bar graph in Figure 4.1. Specimens CC-2.4-20 and UC-1.5-20 reached maximum shears within 1 kip of each other. Specimen UC-1.5-20F experienced a load 7 kips higher than UC-1.5-20. The greater load resisted by UC-1.5-20F demonstrates that the fibers were effective in providing post-cracking resistance to tensile stresses at large deformations, and consequently increased the flexural strength by more than 10% in relation to the two RC specimens.

4.3 Shear vs. Drift

Figures 4.2 and 4.3 show one-to-one comparisons of the measured shear vs. drift ratio curves and Figure 4.4 gathers the envelope curves for all specimens.

In Figure 4.2, the shear vs. drift histories of CC-2.4-20 and UC-1.5-20 are overlaid for comparison. The area enclosed by the hysteretic loops for UC-1.5-20 is clearly smaller than that for CC-1.5-20 due to UC-1.5-20 experiencing lower residual drift ratios (where no shear was applied to the specimen). This characteristic is driven by the reduced amount of flexural reinforcement in UC-1.5-20 in relation to CC-2.4-20. Reduced amounts of
reinforcement lead to reduced cracked moment of inertia. CC-2.4-20 reached peak load during the cycles at 1% drift ratio while UC-1.5-20 reached peak load during the cycles at 2% drift ratio. Both specimens reached similar peak shear during cycles between 2 and 5% drift ratios.

In Figure 4.3, the drift histories of UC-1.5-20 and UC-1.5-20F are overlaid for comparison. The peak shear for UC-1.5-20F was greater than the peak shear for UC-1.5-20 in every cycle. The residual drift ratios that the specimens experienced at zero shear were very similar when the specimens unloaded from nearly identical peak drifts.

Figure 4.4 shows a plot of the overlapped envelope curves for all three specimens. For drift ratios of around 5%, the envelope curve for each specimen was created by plotting the maximum shear against the maximum drift ratio attained in each step of the cyclic loading protocol. The envelope curves support the observations made above about Figure 4.2 and Figure 4.3. For drift ratios exceeding 5%, during the monotonic final push, CC-2.4-20 and UC-1.5-20 had similar slopes in the descending branch with CC-2.4-20 resisting a higher load. Specimen UC-1.5-20F had greater flexural strength and a more gradual loss of strength compared to the two RC specimens.

All specimens completed the cyclic loading protocol (Table 2.2) to drift ratios of 5%. The limiting drift ratio was determined based on a 20% loss of strength (which occurred during the monotonic final push) in relation to the peak load attained in the previous loading step (step 12, 5% drift ratio). Specimen UC-1.5-20F reached 10% drift ratio while still carrying more than 80% of the peak load resisted during the previous loading step. The limiting drift ratios for CC-2.4-20 and UC-1.5-20 were 8% and 7%, respectively, as reported in Tables B.8 and 3.1.

4.4 Longitudinal Strains

Figures 4.5 and 4.6 compare the strains in the longitudinal reinforcing bars at the face of the stub for the controlling column of specimens CC-2.4-20 and UC-1.5-20. The plots show that CC-2.4-20 experienced overall lower longitudinal bar strains at the face of the stub. The strain in the longitudinal bars of CC-2.4-20 decreased as the cycles of the loading protocol progressed past 1.5% drift ratio, it is plausible that the instrumented bars
debonded after the concrete cover spalled. The longitudinal bars of UC-1.5-20 generally reached higher strains with increased drift ratios.

Figures 4.7 and 4.8 compare the strains in the longitudinal reinforcing bars at the face of the stub for the controlling column of specimens UC-1.5-20 and UC-1.5-20F. Careful inspection of the strain data indicates that, for the same drift ratio, longitudinal bars in specimen UC-1.5-20F experienced larger strains. It is plausible that larger strains in UC-1.5-20F are due to the presence of a dominant crack near the face of the stub for the HPFRC specimen.

Figure 4.9 represents the maximum strains in the longitudinal bars measured at various distances from the face of the stub for all three specimens. The data indicate that yield strains are reached at a distance \( d \) (effective depth) away from the face of the support.

**4.5 Curvatures**

Figure 4.10 presents the moment vs. curvature relationships based on strain gage data from the longitudinal bars. The data correspond to the strain gages at the face of the stub of the controlling column for all specimens. The curvatures for specimen CC-2.4-20, in Figure 4.10(a), are limited to values attained during cycles at drift ratios of 2% or less. For all specimens, the curvature increased greatly with little increase in moment once the specimen reached the yield moment. At drift ratios below 2%, where rebar strains are close to yield strains, curvature values are comparable in all three specimens. Figure 4.10(b) includes the data measured during all cycles (up to 5% drift ratio) while Figure 4.10(c) includes data up to 4% drift ratio. The curvatures in specimen UC-1.5-20F are slightly greater than in UC-1.5-20 at drift ratios of 4%, an indication of greater strain concentrations occurring in the HPFRC specimen at large drift ratios. Careful examination of mean curvature data (Figures 3.24 to 3.27 and Figures B.14 to B.15) corroborates these observations.

**4.6 Crack Widths**

Crack width information was only available for specimens UC-1.5-20 and UC-1.5-20F. Figure 4.11 compares the crack widths for these specimens at service load. Service load was
defined as two thirds of the peak load resisted by the control specimen (CC-2.4-20), which occurred approximately at the peak of step 5 (0.6% drift ratio) in both specimens UC-1.5-20 and UC-1.5-20F. At service load, UC-1.5-20 experienced flexural cracks as wide as 0.013 in. and shear cracks as wide as 0.005 in. The averages of the maximum crack widths measured in the north and south column for UC-1.5-20F were 0.008 in. (flexural cracks) and 0.004 in. (shear cracks). The fibers were effective in reducing the overall crack widths at service load.

4.7 Drift Components

The main components of drift considered in this study are drift due to bond slip, shear, and bending. As defined in Chapter 3, drift is the transverse deflection of the column between the points of maximum moment (face of the stub) and zero moment (roller support). Drift ratio refers to the drift divided by the shear span, a, which is 30 in. Drift ratios are reported after adjustments to account for the rotation of the central stub (see Appendix A, section 4.2)

The definition of drift due to bond slip, $\theta_{\text{slip}}$, is illustrated in Figure 4.12. Bond-slip deformations result from the strain gradient along the longitudinal reinforcing bars, from the face of the stub to the center of the stub. To find $\theta_{\text{slip}}$ the strain in the longitudinal reinforcement along the central stub was assumed to vary as shown in Figure 4.13. $\theta_{\text{slip}}$ was calculated by multiplying the average of the strains at the face and center of the stub by half the length of the stub, then dividing by the distance between the compression and tension reinforcement, as expressed in the following equation:

$$\theta_{\text{slip}} = \frac{\varepsilon_{\text{face}} + \varepsilon_{\text{center}}}{2} \left( \frac{L_{\text{stub}}}{2} \right) \frac{L_{\text{stub}}}{d - d'}$$

(4.1)

where $\varepsilon_{\text{face}}$ and $\varepsilon_{\text{center}}$ are the strains in the longitudinal reinforcement of the controlling column at the column-stub interface and at the center of the stub; $L_{\text{stub}}$ is the
length of the stub (16 in.); and \( d-d' \) is the distance between the two layers of reinforcement (8 in.).

Figures 4.14 to 4.16 show the relationship between the strain at the centerline of the stub and the strain at the face of stub for the controlling columns. The target drift for various steps of the loading protocol is labeled on each plot. The strains in the longitudinal reinforcement at the face of the stub in specimen CC-2.4-20 progressively decreased after attaining a drift ratio of 1.5%, potentially due to debonding of the reinforcing bar. According to Figure 4.17, at -2% drift ratio, the contribution of bond slip to drift ratio for CC-2.4-20 reached about 40% of the total drift ratio. Figures 4.18 and 4.19 show that specimens UC-1.5-20 and UC-1.5-20F experienced a lower contribution of bond slip to total drift ratio.

Deformations due to shear were determined using the Whittemore gage measurements as indicated in Section 3.3.6. Figures 3.34, 3.35, and B.17 show the shear contribution to the total drift ratio. The plots show data up to step 9 (2% drift ratio). At drift ratios between 1 and 2%, the contribution of shear deformation to total drift ratio was generally between 10 and 20%, with the HPFRC specimen having near 20% and the RC specimens near 10%. Figures 4.17 to 4.19 show the impact of each of the components assumed to contribute to the total drift ratio for each specimen.

The contribution of flexure to drift ratio was taken by subtracting the contributions of shear and bond slip from the total drift ratio. Figures 4.17 to 4.19 suggest that deformations due to flexure account for 50 to 70% of the total deformation, indicating that deformation capacities calculated based only on curvature distribution should lead to conservative estimates (i.e., actual deformation capacities are 1.4 to 2 times the deformation due to curvature induced by flexure).

### 4.8 Stiffness Reductions

To investigate the influence of the yield strength of longitudinal reinforcement and/or the presence of fibers on the stiffness of the concrete columns, a series of parameters and stiffness definitions (Figure 4.20) were calculated and plotted in Figures 4.21 to 4.26. Figure 4.10 provides definitions for these parameters.
Figure 4.21 shows maximum displacement-to-minimum displacement stiffness ($K_p$) for all specimens at each cycle of the loading protocol. $K_p$ represents the peak-to-peak stiffness. It gives a measure of strength degradation with successive cycles. Generally, $K_p$ decreased for all specimens as the loading protocol progressed. After about the 16th cycle (1.5% drift ratio), $K_p$ was approximately the same for all three specimens. CC-2.4-20 had the highest initial peak-to-peak stiffness, while UC-1.5-20 had the lowest initial peak-to-peak stiffness. The $K_p$ for UC-1.5-20F was similar to CC-2.4-20 throughout the loading protocol. The data in Figure 4.21 suggest that UC-1.5-20 experienced the least strength degradation of the three specimens, while CC-2.4-20 and UC-1.5-20F experienced similar levels of strength degradation.

Figure 4.22 shows the $K_o$-to-$K_r$ ratio for all specimens at the advanced cycles of the loading protocol. $K_o$ is the stiffness measured from zero shear to zero drift ratio, while $K_r$ is measured from zero shear to peak drift ratio. The $K_o$-to-$K_r$ ratio can be used to evaluate any “pinching” effect on the hysteresis that could occur around zero shear, which would be identified typically by a $K_o$-to-$K_r$ ratio of less than 1.0. The lower the $K_o$-to-$K_r$ ratio, the higher the pinching effect. The data in Figure 4.22 show no apparent trends related to the $K_o$-to-$K_r$ ratio. None of the specimens were characterized by values of $K_o$-to-$K_r$ less than 1.0.

Figure 4.23 shows the unloading stiffness ($K_u$) for all specimens. $K_u$ is defined as the stiffness from a peak drift to zero shear. The data in Figure 4.23 correspond to peaks at positive drift ratios. CC-2.4-20 had the highest initial unloading stiffness and, throughout the loading protocol, retained a consistently higher value of $K_u$ when compared to the other specimens. UC-1.5-20F initially had higher values of $K_u$ than UC-1.5-20, but near the 16th cycle these two specimens reached nearly the same $K_u$. Specimens CC-2.4-20 and UC-1.5-20F had comparable $K_u$ during cycles 8 through 16 (0.4 to 1.5% drift ratio).

Figures 4.24 to 4.26 present the data used for deriving the value of the unloading stiffness reduction parameter ($\alpha$). The hysteresis model (Takeda et al., 1970) defines $K_u$ as follows:

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

(4.2)
Equation 4.2 relates the unloading stiffness \( (K_u) \) to the secant stiffness to the yield point \( (K_y) \). \( D_y \) is the displacement at yield, and \( D_{max} \) is the maximum displacement from the displacement history (per direction of loading). To find \( \alpha \), the following equation was used:

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

(4.3)

The points in Figures 4.24 to 4.26 are the averages for the positive and negative direction of loading during the second cycle of the last four steps of the loading protocol. These cycles correspond to drift ratios where the longitudinal bars reached strains exceeding yield. CC-2.4-20 had the lowest value, \( \alpha \approx 0.3 \), while UC-1.5-20 and UC-1.5-20F had similar values, \( \alpha \approx 0.6 \). These values of \( \alpha \) are consistent with the longitudinal reinforcement ratio of the specimens. The unloading stiffness is influenced by the cracked stiffness, which depends on the amount of longitudinal reinforcement.

4.9 Hysteresis Model

The hysteretic behavior of all three specimens may be represented by the Takeda (1970) hysteresis model. A simplified version of the Takeda model is illustrated in Figure 4.27, based on a bilinear primary curve omitting the cracking point. In addition to the unloading stiffness reduction parameter \( (\alpha) \), the model is mainly defined by the initial stiffness \( (K_y) \), the shear at yield \( (V_y) \), and the post-yield stiffness \( (K_{py}) \).

The model assumes a force-displacement response with an initial slope (from the origin) of \( K_y \) up to the yield point \( (D_y, V_y) \). After yielding, the post-yield stiffness \( (K_{py}) \) is used as the slope to the maximum displacement, at which point the unloading stiffness \( (K_u) \) is applied from the point of maximum displacement to zero shear. The stiffness at reloading (from zero shear) is defined by targeting the point of maximum displacement (per direction). In the calculations presented below, the value of \( K_u \), as defined in equation 4.2, was determined using a different unloading stiffness reduction coefficient \( (\alpha) \) for each specimen, derived from Figures 4.24 to 4.26.
Figures 4.28 to 4.30 show the shear vs. drift ratio curves for each specimen overlaid with the Takeda hysteresis model. The first segment of the calculated values is associated with the yield point. Thereafter, the Takeda hysteresis data are derived using target drift ratios of 1.5, 2, 3, and 4%. The last cycle, with a target drift ratio of 5%, are based on the actual maximum drift ratio attained. The data in the figures show that the Takeda hysteresis model is a reasonable approximation to the overall behavior.

4.10 Simplified Method for Calculation of Strength using ACI 318 Code

The maximum measured moment was compared to the moment calculated using the general principles outlined in ACI 318-08 (Table 4.1). Calculated data are presented for the nominal flexural strength \( M_n \) and probable flexural strength \( M_{pr} \). Values of \( M_n \) were obtained using an ultimate compressive strain of concrete, \( \varepsilon_u \), of 0.003 and the specified yield strength, \( f_y \). Values of \( M_{pr} \) were based on \( \varepsilon_u = 0.004 \) and \( 1.25 f_y \). The increased value of \( \varepsilon_u \) to 0.004 when calculating \( M_{pr} \) allows the development of higher stresses in the compression reinforcement. The compressive strength of the concrete \( f'c \), was based on the values reported in Tables A.1 and B.6. The calculated moments represent the flexural strength associated with an axial load of \( 0.2 f'c A_g \).

Comparison of the measured vs. calculated data in Table 4.1 indicates that the calculated values of \( M_n \) are lower (conservative) in relation to the maximum measured moments. The calculated values of \( M_{pr} \) were within 10% of the measured maxima, indicating reasonable agreement.
Chapter 5. Summary and Conclusions

5.1 Summary

The aim of this investigation was to study the cyclic behavior of concrete members (with and without fibers) reinforced longitudinally with SAS 670 Grade-97 steel bars, with a sharp yield point and a tensile strength-to-yield strength ratio of 1.2. The main focus was on laboratory experiments developed to evaluate the deformation capacity and hysteretic response of concrete columns subjected to reversed-cyclic transverse loading and reinforced longitudinally with high-strength steel bars.

Chapter 1 includes a summary of previous research relevant to the use of high-strength reinforcement in concrete members with emphasis on experimental programs considering the use of ultrahigh strength steel (with yield strengths greater than 80 ksi) and high-performance fiber reinforced concrete (HPFRC). In this study, HPFRC was defined as a class of fiber reinforced concrete, which exhibits tensile strain-hardening behavior of the concrete matrix after first cracking and involving multiple cracks at high strain levels.

Chapter 2 defined the experimental program, which involved two specimens reinforced with Grade-97 steel bars. Another specimen (tested by Rautenberg, 2011) acted as the control specimen reinforced with conventional Grade-60 steel bars. Each specimen consisted of two cantilever columns spanning in opposite directions from a common central stub. The central stub was attached to a hydraulic cylinder mounted within a self-reacting loading frame. The target loading protocol, patterned after the recommendations of FEMA 461, included two full cycles at each displacement step, with drift ratios ranging from 0.15% to 5% in 12 steps. The drift ratios were determined using the transverse displacement of the central stub divided by the column shear span with adjustments to account for the rotation of the central stub. The axial load, applied using four external post-tensioning bars, was kept constant throughout the loading protocol. After completion of the 24-cycle loading protocol (two cycles in each of the 12 steps), the specimens were subjected to monotonically increasing displacements until failure of the specimen or up to displacement limits of the testing apparatus.
The following parameter values define the two specimens reinforced with Grade-97 steel bars:

**Column dimensions**
- **Width, b:** 9 in.
- **Depth, h:** 12 in.
- **Effective depth, d:** 10 in.
- **Shear span, a:** 30 in.

**Concrete matrix**
- **Specified compressive strength, \( f'c \):** 6000 psi
- **Proportions by weight (cement : fine : coarse):**
  - \( 1.0 : 3.4 : 4.4 \) (RC)
  - \( 1.0 : 2.0 : 1.4 \) (HPFRC)
- **Maximum aggregate size:**
  - 1.0 in. (RC)
  - 0.5 in. (HPFRC)
- **Fiber volume fraction, \( V_f \):**
  - 0% (RC)
  - 1.5% (HPFRC)
- **Fiber type (hooked steel fibers):**
  - RC-80/30-BP (Dramix by Bekaert)

**Longitudinal reinforcement**
- **Specified yield strength, \( f_y \):** 97 ksi
- **Bar diameter:** 0.71 in. (18 mm)
- **Bar layout:** Symmetrical (4 bars)
- **Reinforcement ratio, \( \rho_g \):** 1.5%

**Transverse reinforcement**
- **Specified yield strength, \( f_{yt} \):** 60 ksi
- **Type:** #3 rectangular hoops
- **Spacing, s:**
  - \( d/4 \) (RC)
  - \( d/2 \) (HPFRC)
- **Reinforcement ratio, \( \rho_t \):**
  - 0.98% (RC)
  - 0.49% (HPFRC)
Applied loads (based on measured $f'c$)

\[
\begin{align*}
\text{Axial} & \quad 0.2 f'c \cdot A_g \\
\text{Shear} & \quad 6.6\sqrt{f'c[\text{psi}]}b \cdot d
\end{align*}
\]

With the inclusion of specimen CC-2.4-20 (presented in Appendix B) tested by Rautenberg (2011), the main variables of this study were:

Specified yield strength of longitudinal reinforcement, $f_y$:

- 60 ksi (specimen CC-2.4-20, $\rho_g = 2.4\%$)
- 97 ksi (specimens UC-1.5-20 and UC-1.5-20F, $\rho_g = 1.5\%$)

Fiber volume fraction, $V_f$:

- 0% (specimens CC-2.4-20 and UC-1.5-20)
- 1.5% (specimen UC-1.5-20F)

Spacing of transverse reinforcement, $s$:

- $d/4$ (specimens CC-2.4-20 and UC-1.5-20)
- $d/2$ (specimen UC-1.5-20F)

Chapter 3 reported the measured experimental data: strains in the longitudinal and transverse reinforcement, applied axial force and displacements, Whittemore gage readings, and width of cracks. Measurements of drift ratio, transverse load, axial load, and strains in both longitudinal and transverse reinforcement were taken continuously during each test. Whittemore gage data were recorded at the second-cycle peaks for each of the 12 steps of the loading protocol and at the end of each step (zero transverse load). Crack widths were measured at the peaks of each loading cycle.

Chapter 4 discussed and compared the measured response and information derived from the measured data, such as curvatures, components of drift, and stiffness. The measured force-displacement data were also compared with the calculated response based on the Takeda hysteresis model. Parameter values were recommended for the
application of the Takeda model to concrete columns reinforced with SAS 670 Grade-97 steel bars. The chapter included a method for calculating the nominal and probable moments in good agreement with the measured moments.

Appendix A describes the material properties, specimen dimensions and instrumentation, test setup, data acquisition, load application, and test procedure. The measured compressive strength of concrete in the specimens ranged between 4790 and 6600 psi. The measured steel properties for the Grade-97 bars were: $f_y = 97.3$ ksi, $\epsilon_{su} = 10.4\%$, and $f_u / f_y = 1.2$. Appendix B contains a summary of the relevant data (similar to Appendix A) and test results (similar to Chapter 3) for the control specimen CC-2.4-20 (tested by Rautenberg, 2011), reinforced with conventional Grade-60 steel bars.

5.2 Conclusions

The following findings are drawn from the test results and within the scope of parameter values presented in Section 5.1:

On drift capacity:

1. For concrete columns (RC and HPFRC), replacing conventional Grade-60 longitudinal reinforcement with reduced amounts of Grade-97 steel bars led to comparable drift capacity. The reinforcement was reduced in proportion to the increase in yield strength. All specimens were capable of retaining more than 80% of their capacity (measured maximum force) throughout the cyclic loading protocol up to drift ratios of 5%.

2. For the HPFRC column, using reduced amounts of transverse reinforcement (by increasing the hoop spacing from $d/4$ to $d/2$) did not reduce the maximum usable drift capacity in relation to the RC columns with twice the amount of hoops.

On flexural strength:

3. For RC columns, replacing conventional Grade-60 longitudinal reinforcement with reduced amounts of Grade-97 steel bars led to nearly identical flexural capacity. The reinforcement was reduced in proportion to the increase in yield
strength. The presence of fibers in the HPFRC column increased the flexural strength by approximately 10% in relation to the RC columns.

4. For RC columns reinforced with Grade-97 steel bars, the maximum probable moment calculated using a rectangular compressive stress block for the concrete with ultimate strain of 0.004 and assuming an elastoplastic model for the steel with a maximum stress of 1.25fy, was in good agreement with the maximum moment resisted by the test column. For HPFRC columns, the maximum probable moment may be determined using 1.1 times the calculated value for RC columns.

On stiffness:

5. Using proportionally reduced amounts of Grade-97 steel bars in place of Grade-60 longitudinal reinforcement led to reduced post-cracking stiffness and greater yield deformations. For RC and HPFRC columns reinforced with Grade-97 steel bars, the unloading stiffness at drift ratios of 2% was approximately 60% and 70%, respectively, of the unloading stiffness measured in the RC column with Grade-60 bars.

On crack width:

6. The HPFRC column was effective in reducing crack widths. At the reference service load level (acting in the transverse direction), the HPFRC column with Grade-97 steel bars experienced maximum crack widths of 0.008 in. while the RC column with Grade-97 bars experience crack widths of up to 0.013 in.

The test results suggest that the use of high-strength Grade-97 steel bars as concrete reinforcement is a viable option for columns in earthquake-resistant construction. However, it is important to note that the specimens presented in this study were designed to have flexural yielding dominate the nonlinear cyclic response. The specimens were designed and detailed to avoid failures due to bond, shear, bar buckling, or beam-column joint distress. Further studies with realistic moment-frame configurations are needed to identify conditions likely to cause considerable loss of strength and stiffness during strong ground motions.
Tables
Table 2.1 – Specimen Description

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Concrete Strength, $f'c$, psi</th>
<th>$\frac{P}{f'cA_g}$</th>
<th>Volume of Fibers, %</th>
<th>Longitudinal Reinforcement</th>
<th>Transverse Reinforcement</th>
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<tbody>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>Top</td>
<td>Bottom</td>
</tr>
<tr>
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<td>4790</td>
<td>20</td>
<td>0</td>
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<td>2#6</td>
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<tr>
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<td>6600</td>
<td>20</td>
<td>1.5</td>
<td>2#6</td>
<td>2#6</td>
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</table>

1 Control specimen CC-2.4-20 is described in Appendix B. See Table B.6.
2 SAS 670 threaded bar. Bar diameter is 0.7 in. (18 mm) with an area of 0.39 in.² (250 mm²).
3 The specified volume fraction of fibers exceeds the critical volume fraction that warrants strain-hardening behavior in tension after first cracking.

Table 2.2 – Loading Protocol

<table>
<thead>
<tr>
<th>Step¹</th>
<th>Drift, %²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.15</td>
</tr>
<tr>
<td>2</td>
<td>0.2</td>
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</tr>
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<td>4.0</td>
</tr>
<tr>
<td>12</td>
<td>5.0</td>
</tr>
</tbody>
</table>

¹ Each step includes two cycles.
Table 3.1 – Maximum Measured Drift and Shear

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum Drift, %</th>
<th>Maximum Shear, V&lt;sub&gt;max&lt;/sub&gt; kip</th>
<th>&lt;br&gt; v&lt;sub&gt;max&lt;/sub&gt; = V&lt;sub&gt;max&lt;/sub&gt; b d / \sqrt{f'c}^2</th>
<th>&lt;br&gt; v&lt;sub&gt;c&lt;/sub&gt; = \frac{V&lt;sub&gt;max&lt;/sub&gt; - V_s}{b d \sqrt{f'c}}</th>
</tr>
</thead>
<tbody>
<tr>
<td>UC-1.5-20</td>
<td>7.0</td>
<td>41</td>
<td>6.6</td>
<td>0</td>
</tr>
<tr>
<td>UC-1.5-20F</td>
<td>10.0</td>
<td>48</td>
<td>6.6</td>
<td>2.5</td>
</tr>
</tbody>
</table>

1 Load carrying capacity of the specimens was maintained (within 80% of the peak load reached in the first cycle of the previous loading step) throughout the loading protocol and into the final monotonic push. Reported drift ratios were limited to a maximum of 10% due to sensor limitations.

2 Based on f'c (psi) at test date (see Table A.1), b = 9 in., d = 10 in.

3 v<sub>c</sub> is the shear stress attributed to concrete assuming the contribution of transverse steel reinforcement is given by V<sub>s</sub> = A<sub>v</sub>f<sub>y</sub>d/s, where A<sub>v</sub> = 0.22 in<sup>2</sup>, f<sub>y</sub> = 66.9 ksi (see Table A.3), and d/s of 4 for UC-1.5-20 and 2 for UC-1.5-20F.

Table 3.2 – Measured Yield Point

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Shear&lt;sup&gt;1&lt;/sup&gt; kip</th>
<th>Drift ratio %</th>
<th>Secant stiffness&lt;sup&gt;2&lt;/sup&gt; kip/in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean</td>
<td>Mean</td>
</tr>
<tr>
<td>UC-1.5-20</td>
<td>South +</td>
<td>29.4</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td>South -</td>
<td>34.1</td>
<td>1.45</td>
</tr>
<tr>
<td></td>
<td>North +</td>
<td>35.8</td>
<td>1.38</td>
</tr>
<tr>
<td></td>
<td>North -</td>
<td>33.2</td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td>South +</td>
<td>42.5</td>
<td>1.20</td>
</tr>
<tr>
<td></td>
<td>South -</td>
<td>36.9</td>
<td>0.87</td>
</tr>
<tr>
<td></td>
<td>North +</td>
<td>41.3</td>
<td>1.14</td>
</tr>
<tr>
<td></td>
<td>North -</td>
<td>38.3</td>
<td>1.23</td>
</tr>
</tbody>
</table>

1 Shear force measured at first yield of the longitudinal reinforcement. Top and bottom bars were instrumented with strain gages at the north and south faces of the central stub. The yield strength of the longitudinal reinforcing bar is based on the 0.2%-offset method, see Table A.3. The yield point accounts for the initial axial strain induced by the applied axial load, P. The initial strain was estimated using P/(A<sub>g</sub> E<sub>c</sub>), where E<sub>c</sub> was based on 57000 \sqrt{f'c [psi]} and A<sub>g</sub> is the gross area of the cross section (b h).

2 Secant stiffness to first yield of the longitudinal reinforcement.
Table 4.1 – Calculated and Measured Maximum Moments

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Calculated Moment(^1)</th>
<th>Measured Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(M_n, \text{kip-ft})</td>
<td>(M_{pr}, \text{kip-ft})</td>
</tr>
<tr>
<td>CC-2.4-20</td>
<td>94</td>
<td>107</td>
</tr>
<tr>
<td>UC-1.5-20</td>
<td>85</td>
<td>97</td>
</tr>
<tr>
<td>UC-1.5-20F</td>
<td>99</td>
<td>111</td>
</tr>
</tbody>
</table>

\(^1\) Based on general principles outlined in ACI 318-08. \(M_n\) is based on \(\varepsilon_u = 0.003\) and \(f_y\); \(M_{pr}\) is based on \(\varepsilon_u = 0.004\) and \(1.25 f_y\), where \(f_y\) is the specified yield strength. The value of \(f'c\) is based on the values reported in Tables A.1 and B.6. Calculated moments represent the flexural strength associated with an axial load of \(0.2 f'c \cdot A_g\). The values of \(P\) were 113 k, 103 k, and 143 k for CC-2.4-20, UC-1.5-20, and UC-1.5-20F, respectively. Calculations did not include the effect of fibers.
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Step 9 (2% Drift Ratio), First Peak  
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Step 12 (5% Drift Ratio), First Peak

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Figure 3.13—Continued
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Figure 3.14—Continued
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Figure 3.15—Continued
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Figure 4.26—Derived Unloading Stiffness-Reduction Parameter, $\alpha$, for Specimen UC-1.5-20F
Figure 4.27—Takeda Hysteresis Model, Pfund (2012)

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

Figure 4.28—Cyclic Response of Specimen CC-2.4-20, Takeda Hysteresis vs. Measured Response

Hysteresis Parameters:

- \( V_y = 40 \text{ kip} \)
- \( D_y = 0.90 \% \)
- \( k_{py} = 0 \)
- \( \alpha = 0.35 \)
Figure 4.29—Cyclic Response of Specimen UC-1.5-20, Takeda Hysteresis vs. Measured Response

Figure 4.30—Cyclic Response of Specimen UC-1.5-20F, Takeda Hysteresis vs. Measured Response
Appendix A. Materials and Test Setup

A.1 Introduction

Appendix A describes the process of constructing and testing the specimens reinforced with high-strength steel bars. The following sections contain information on specified material properties and nominal specimen dimensions, as well as actual measured values. Additionally, this appendix describes the specimen instrumentation and testing procedure.

A.2 Materials

A.2.1 Concrete

The specimens were cast on two separate days, one week apart. Casting day information as well as test-day compressive strength can be found in Table A.1. Centre Concrete Company in State College, Pennsylvania, provided concrete for the specimens. The concrete mix proportions are shown in Table A.2. The 28-day compressive strength was specified as 6000 psi.

Aggregate size was limited to a maximum ½ in. for specimen UC-1.5-20F to facilitate mixing, dispersion, and consolidation of the steel fibers in the concrete. Aggregate size in specimen UC-1.5-20 was limited to a maximum of 1 in. To improve concrete workability, a water-reducing agent (Glenium 3030NS) was added to the concrete on site. The 1.5% volume fraction of hooked steel fibers was also added to the concrete on site before casting. Bekaert Corporation provided the Dramix RC-80/30-BP hooked steel fibers, which have a length of 1.2 in. and an aspect ratio (length to diameter) of 80.

Concrete compressive strength for both specimens was found via testing of 4-in. diameter, 8-in. long concrete cylinders cast alongside the specimens. These cylinders were tested following ASTM C39 at 3, 7, 14 and 28 days after casting. The results of the cylinder tests are reported in Figures A.1 and A.2. Figure A.3 shows cylinder specimens at the end of compression tests, where the noted specimen name “UC4-X” refers to UC-1.5-20, and “UC2-F” refers to UC-1.5-20F. Additional cylinders were tested on the first day of
application of the loading protocol, defining the “test date” compressive strength reported in Table A.1.

The concrete modulus of rupture, $f_r$, reported in Table A.1, was found through the testing of 4 in. by 4 in. by 14 in. beams that were cast alongside the specimens and cylinders. Following ASTM C78, these beams were subjected to third-point load flexural testing on the last day of application of the loading protocol for each specimen, see Figure A.4.

A.2.2 Reinforcement

Each specimen contained the same type of longitudinal reinforcement and transverse reinforcement, thus yielding the first two letters in the specimen nomenclature: UC. The longitudinal bars for both specimens (Table 2.1), classified under “U” for “ultrahigh” strength steel, were SAS 670 Grade 97 bars with a diameter of 18 mm (approximating a #6 bar). The transverse reinforcement for both specimens consisted of #3 Grade 60 hoops, designated as “C” for “conventional” strength steel.

SAS Stresssteel, Inc., provided the Grade 97 longitudinal bars (SAS Threadbar Systems). The yield strength, average tensile strength, and ultimate elongation for these threaded bars were reported by Tavallali (2011), see Table A.3. These values were obtained via tests following ASTM A370 using three randomly sampled coupons. The resulting stress-strain curves and yield points, determined using the ASTM A370 recommended 0.2%-offset method, are shown in Figure A.5. The tests resulted in an average yield strength $f_y = 97.3$ ksi, an average tensile strength $f_u = 117$ ksi, and an ultimate elongation $\epsilon_{su} = 10.4\%$. The $f_u/f_y$ ratio is 1.20, which is below the minimum requirement of 1.25 for seismic applications as recommended in ACI 318 (2008).

Sheesley Concrete Supply Company in State College, Pennsylvania, provided the transverse reinforcement used in the specimens. All transverse reinforcement was fabricated as hoops with seismic hooks at both ends. All of the hoops were taken from a single heat, resulting in all the hoops sharing the same mill certificate. Material properties for the steel used for the hoops are reported in Table A.3. The transverse reinforcement consisted of #3 ASTM A615 Grade 60 deformed bars.
A.3 Specimens

Each of the specimens consisted of two columns connected to a central stub, with the column ends supported 30 in. from the stub face, at the assumed inflection point, by rollers. The loading protocol was applied through this central stub, subjecting the specimen to single-curvature bending. The behavior of each column was assumed to be independent of the opposing column, with the stub acting as a fixed base for the cantilevered columns. The lateral load was applied to the central stub via a hydraulic actuator, thus loading both column elements simultaneously. A schematic view of the front and side of the test apparatus can be found in Figure A.6. The specimens were also loaded with a constant, concentric axial load of \( 0.2 f_c \cdot A_g \) throughout the duration of the test via an external post-tensioning system, see Figure A.7. The second numerical identifier in the specimen names, “20” refers to the axial load, 20% of \( f_c \cdot A_g \).

A.3.1 As-Built Dimensions

As-built dimensions for the specimens are reported in Tables A.4 and A.5, in relation to Figure A.8. The as-built dimensions were found by measuring the formwork prior to casting.

A.3.2 Reinforcement Details

Figures 2.2 and A.9 show the reinforcement details for both specimens. The first numerical identifier in the specimen nomenclature, “1.5”, identifies the longitudinal reinforcement ratio, \( \rho_g \), used for both specimens. The deformations on the longitudinal bars used in the specimens had a threaded pattern with comparable bond characteristics to conventional bars according to CTL Group (2005). Longitudinal reinforcement was continuous through the length of the specimen and had a 90° hook at each end.

The transverse hoops were fabricated with seismic hooks at both ends, as shown in Figure A.9, following Chapter 21 of ACI 318 (2008). Hoops were nominally spaced at \( d/4 \), or 2.5 in., for specimen UC-1.5-20, and proportioned to satisfy shear strength requirements following Chapter 21 of ACI 318 (2008). Hoops were nominally spaced at \( d/2 \), or 5 in., for the HPFRC specimen UC-1.5-20F. The first hoop in each column closest to the central stub
was located between 1 and 1.5 in. from the stub face, varying due to construction tolerances. The transverse reinforcement ratio, $\rho_t$, was 0.98% for UC-1.5-20, and 0.49% for UC-1.5-20F.

### A.3.3 Formwork, Casting and Curing

Segmented wood forms were designed for the specimen dimensions with the aim of easy assembly and disassembly, as shown in Figures A.10 and A.11. The steel reinforcement was assembled into cages and then instrumented with strain gages before being placed in the formworks. The assembled reinforcement cages can be seen in Figure A.12. A few days prior to casting, the interiors of the formworks were coated with form oil before the reinforcement was placed inside. Concrete was placed into the formworks directly from the chute of the truck, and then vibrated to remove air pockets, as shown in Figure A.13. To improve workability, specimen UC-1.5-20F required 20% more water reducing agent (Glenium 3030NS) than in specimen UC-1.5-20. Concrete for the cylinders and flexural test beams was shoveled into place from a wheelbarrow. All exposed surfaces were leveled and finished using a float and trowel, as shown in Figures A.14 and A.15. The concrete was then covered with wet burlap and plastic sheeting for curing. The burlap was re-watered daily for the first three days. Curing followed ASTM C31.

Forms were removed after 7 days. The specimens were stored in the laboratory until they were tested. Cylinders were tested for compressive strength at 3, 7, 14, and 28 days after casting, as well as on the day the corresponding specimen began the testing protocol. Refer to Figure A.1, Figure A.2, and Table A.1.

### A.4 Test Procedure

This research program investigated the behavior of concrete columns reinforced with high-strength steel bars subjected to displacement reversals. The loading protocol used in the experiments followed the recommendations in FEMA 461 for quasi-static cyclic testing of structural components. The loading protocol provided repeating cycles of step-wise increasing deformation amplitudes, with the specimen being loaded to each amplitude twice. For each successive step of two cycles at a single amplitude, the target
deformation amplitude increased by approximately 40%. At the end of the testing protocol, summarized in Table 2.2 and Figure 2.3, each specimen underwent a monotonic push until the specimen reached failure or the test apparatus limits.

A.4.1 Test Setup

The specimens were tested in an upright self-reacting steel frame, pictured in Figure A.6, with four roller supports near the column ends, located at the assumed inflection points. Details on the design of the loading frame can be found in Tavallali (2011). The established loading protocol was administered through the central stub of the specimen via an Enerpac RR-3006 double-acting hydraulic cylinder (attached to the top horizontal member of the reaction frame with bolts), which was manually operated by the administrator of the test. The connection of the specimen stub to the loading cylinder was pinned to avoid rotational restraint of the central stub (refer to Figure 2.5). Further information about this connection can also be found in Tavallali (2011).

Each specimen was placed in the facility on the lower roller supports, as shown in Figure A.16. Then, spacers and endplates were adhered to the ends of the columns, as seen in Figure A.17. These endplates, detailed in Figure A.18, facilitated the application of a concentric axial load using a roller, with minimal moment transfer to the specimen. The central stub of the specimen was aligned in the testing frame and clamped to the pin connection of the loading cylinder through the use of an upper and lower plate, and threaded bars and nuts. Then, the upper roller supports were placed near the column ends and the roller plates were held in place 1/8 in. above the specimen through the use of threaded bars and nuts.

Next, the post-tensioning system used to apply axial load was assembled. This post-tensioning system, shown in Figure A.7, included four 1.5-in. diameter steel bars, with a yield strength $f_y = 125$ ksi, running through two 1.5-in. thick steel plates, A572 Gr. 60. These steel plates sandwiched the specimen through contact with the endplates. Details for these elements can be found in Figure A.18. Miller Welding Services of State College, Pennsylvania, provided all plates, rollers, and post-tensioning bars. A Powerteam RH303 hollow-centered double-acting hydraulic cylinder was placed at one end of each bar, to rest against one of the large steel plates. A ¼”x4”x4” plate and a 1-5/8” hex-nut functioned
as a reaction surface for the hydraulic cylinder on each bar. Each cylinder was individually
controlled by manifolds on both the advance and return sides of a 10,000-psi pump, which
was manually operated to maintain axial load in the specimen through the duration of the
test.

The total weight of the specimen including the steel plate attachments and the post-
tensioning system was approximately 2.5 kips. This weight is the initial force in the
actuator after fully lifting the specimen (pulling direction). The recorded data include this
weight without adjustments.

A.4.2 Measurements

Measurements were taken throughout the tests to monitor the following: lateral
load applied to the specimen stub, axial load on the specimen, drift ratio of the columns,
size of cracks, strains on the reinforcing bars, and concrete surface strains. All strain gages
had a specified resistance of 120 Ω, and all were provided by Texas Measurements, Inc.

The applied transverse load was plotted against the adjusted drift ratio for both
columns of each specimen. These adjusted drifts were calculated using data from
displacement sensors that measured specimen movements. Displacement sensors were
located at stub-column interfaces and at the support locations, as shown in Figure 2.6. The
sensors consisted of linear potentiometers with a range of ±2 in. at the interfaces and
±0.25 in. at the supports. The drift ratio, \( \delta \), was calculated from the following base
equation:

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

(A.1)

where \( \Delta \) is the deflection of the specimen stub, taken as the average of the readings from
the four potentiometers located at the stub faces, and \( a \) is the shear span, taken as the
distance from the face of the stub to the roller support. Equation A.1 was adjusted to
account for stub rotation, using the following:
\[ \delta_{adj} = \frac{\Delta \pm w\beta}{a} \pm \beta \]  

(A.2)

where \( w \) is the actual distance from the centerline of the stub to the stub face, and \( \beta \) is the stub rotation in radians. The value of \( \delta_{adj} \) measures the adjusted drift ratio for the individual columns of the specimen. Stub rotation was calculated using all linear potentiometer readings. Figure A.20 shows the dimensions used for the calculation of column drift ratios.

Transverse load was measured using a custom load cell attached to the platen assembly adapted to the ram of the Enerpac RR-3006 hydraulic cylinder, as shown in Figure 2.5. This platen assembly was designed to remain in the elastic range for the load range used in these tests, providing a linear relationship between load and strain. The load cell uses FLA-3-5L Texas Measurements strain gages with a 2% strain rating and was calibrated against a MTS 661.23A-02 load cell, over a range of ±100 kip.

Axial load applied to the specimen was measured individually on each of the four post-tensioning bars used to apply load. Each bar was instrumented with six of the aforementioned FLA-3-5L strain gages, split into two groups of three gages: one group of gages aligned perpendicular to the direction of the bar, and one group of gages aligned parallel to the direction of the bar. The gages in each group were arranged 120° apart along the circumference of the bar and wired in series. Gages were protected in accordance with recommendations by Texas Measurements, Inc. The two groups of gages wired in series for each bar were connected in a half-bridge configuration to a Measurements Group 2120 strain amplifier whose output was routed to the data acquisition system. Axial load was monitored continuously during the tests with separate readouts for each bar and another for the sum of all four bar loads, i.e., the total axial load applied to the specimen.

Strain in the column reinforcing bars was monitored at selected points along each bar and on two hoops of each column. A total of 24 strain gages were used on each specimen as shown in Figure 2.7. The strain graphs in Chapters 3 and 4 correspond to selected gages. Different types of strain gages were applied to the longitudinal reinforcement and the transverse reinforcement. YEFLA-5-5L gages rated for a 10%
maximum strain were applied to longitudinal reinforcement. FLA-3-5L gages rated for 2% maximum strain were applied to transverse reinforcement. Strain gages were all wired into either Measurements Group 2120 strain amplifiers or National Instruments SCXI 1520 strain amplifiers in a quarter-bridge configuration. The strain amplifier outputs along with other conditioned sensor outputs were passed to a National Instruments PCI 6021 data acquisition board to facilitate real-time recording, processing, and reporting of sensor data in a custom LabVIEW software interface.

Crack widths were measured with a crack comparator and recorded manually at the peak of each loading cycle.

Finally, measurements of all horizontal, vertical, and diagonal dimensions in the Whittemore grid, shown in Figure 2.8, were taken using Linear Variable Differential Transformers (LVDTs) with a range of ±0.25 in. Two different electronic Whittemore gage instruments (Figure A.21) were used—one for the 8 in. vertical and horizontal elements in the grid, and one for the 11.2 in. diagonal elements. For every step of the loading protocol, the Whittemore data were taken at the third and fourth peak, and at the end of the load step corresponding to zero applied transverse load. These measurements were used to determine the component of drift due to shear for each specimen, as explained in Chapter 4.

A.4.3 Testing Sequence

The testing sequence used with specimens UC-1.5-20 and UC-1.5-20F was very similar to the testing sequence used in previous experiments on beam specimens with the same loading frame, as explained in Pfund (2012) and Tavallali (2011). The list below, taken from Pfund (2012), details the step-by-step sequence. In addition to what is included in the step-by-step outline for beam tests, axial load was carefully monitored and adjusted throughout the column tests. The test administrator monitored the axial load on all four bars individually, and would call out which bar to adjust, and how to adjust it, continuously throughout the test. Another individual, who assisted with the test, would then adjust the axial load in that bar to the appropriate level. The axial load was adjusted when either of the preset limits was exceeded. The overall axial load variation was limited to a range of ±3% of the total target load; the axial load variation per bar was limited to a
range of ±10% of the target load (equivalent to ±2.5% of the total axial load provided by four bars).

Listed below is the testing sequence used in the column tests:

I. Start data acquisition system (DAS) and apply axial load
II. Push specimen to positive peak (first cycle)
   a. Close pump valves
   b. Stop DAS
   c. Mark and record size of cracks
   d. Change whiteboard information (step, peak, drift, and load)
   e. Take photos (one of full specimen and one close up between end supports)
   f. Resume DAS, then open pump valves
III. Unload to zero transverse load
   a. Change whiteboard information
   b. Take photos
IV. Pull to negative peak
   a. Repeat II
V. Unload to zero transverse load
   a. Repeat III
VI. Push to positive peak (second cycle)
   a. Close pump valves
   b. Stop DAS
   c. Mark and record size of cracks
   d. Change whiteboard information (peak, drift, and load)
   e. Take photos (one of full specimen and one close up between end supports)
   f. Record Whittemore measurements in DAS
      i. Take calibration-plate measurements before and after taking the set of face measurements
   g. Resume DAS, then open pump valves
VII. Unload to zero transverse load
   a. Repeat III
VIII. Pull to negative peak
   a. Repeat VI
IX. Unload to zero transverse load
   a. Repeat III
X. Record Whittemore measurements in DAS to complete step 1 of loading protocol
XI. Repeat II through X for all 12 steps of loading protocol (Table 2.2)
XII. Remove support sensors and reset face sensors to 4.0 in.
XIII. Acquire new baseline and run final monotonic push to 10% drift (or until specimen fails)
XIV. Adjust data obtained from the final push by adding residual data from the end of the loading protocol
Appendix B. Column Specimen CC-2.4-20

B.1 Introduction

Appendix B describes the control specimen, CC-2.4-20, which was constructed and tested by Rautenberg (2011). The following sections present information on the material properties, dimensions, test setup, and instrumentation for specimen CC-2.4-20. Data are also presented for the measured response: applied shears, drift ratios, strains, mean curvatures, and deformation components. The response comparison and discussion of specimens UC-1.5-20, UC-1.5-20F, and CC-2.4-20 are presented in Chapter 4 of this study. Data included in this appendix were taken from Rautenberg (2011) and have been edited to show only information concerning CC-2.4-20.

B.2 Materials

This section presents information about the material properties of the concrete and steel reinforcement used for specimen CC-2.4-20 (Rautenberg, 2011).

B.2.1 Concrete

Of the six concrete batches used to cast the specimens tested by Rautenberg (2011), CC-2.4-20 was cast using concrete from batch designation C03. Mix proportions for concrete batch C03 are shown in Table B.1. Compressive strength was tested using 6-in. diameter by 12-in. long cylinders, and following ASTM C39. Modulus of rupture was found following AST C78 by testing 6-in. square by 22-in. long beams using third-point loading over an 18-in. span. Final flexural and compressive properties of batch C03 are presented in Table B.2. The flexural tests were only conducted on the first day of application of the loading protocol (test day). The compressive strength tests were typically conducted at 1, 3, 7, 14, 21, and 28 days after casting, as well as on test day. Table B.3 presents the individual and average compressive strengths for the cylinders tested over time.
B.2.2 Reinforcement

Of the eleven heats of steel used as reinforcement in the specimens tested by Rautenberg (2011), CC-2.4-20 used heat designation S08 for longitudinal steel and heat designation S11 for transverse steel, as shown in Table B.4. Table B.5 presents the properties of these two heats, which were found through tensile tests of coupons with 8-in. gage length following ASTM A370. The longitudinal reinforcing bars in specimen CC-2.4-20 were #6 deformed Gr. 60 with an average yield strength of 65 ksi, and an average tensile strength of 91 ksi. An ultimate elongation of 16% was measured in 8-in. gage length. The transverse reinforcing bars were #3 deformed Gr. 60 with an average yield strength of 62 ksi, and an average tensile strength of 92 ksi. The Gr. 60 reinforcing bars used in specimen CC-2.4-20 comply with ASTM A706.

B.3 Specimen

Specimen CC-2.4-20 was very similar to the specimens described in Appendix A. The specimen consisted of a central stub with two columns extending from its opposite sides, supported by rollers at 30 in. away from the stub face. The loading protocol was administered through the central stub via a hydraulic actuator while the specimen experienced a concentric axial load of 0.2 $f'c \cdot A_g$ throughout the duration of the test via an external post-tensioning system. The central stub for specimen CC-2.4-20 is 4 in. shorter than the stub in specimens UC-1.5-20 and UC-1.5-20F due to dimensions associated with the attachment of the roller supports to the loading frame at Penn State. Additionally, CC-2.4-20 was not tested in a vertical loading frame, but instead, horizontally. These differences were assumed to not cause noticeable changes in specimen response. Overall information about specimen CC-2.4-20 can be found in Table B.6, and additional details can be found in section A.3 of Rautenberg (2011).

B.3.1 As-Built Dimensions

Nominal dimensions for specimen CC-2.4-20 can be found in Figure B.1. The column cross-section dimensions were the same as the dimensions for specimens UC-1.5-20 and
UC-1.5-20F, 9-in. wide by 12-in. deep. The shear span, \( a \), was 30 in. The central stub length was 12 in. As-built dimensions were within 0.25 in. of nominal values.

**B.3.2 Reinforcement Details**

Figure B.2 shows the reinforcement details for specimen CC-2.4-20. The longitudinal reinforcement ratio, \( \rho \), was 2.4%. The longitudinal reinforcing bars, continuous through the central stub, were #6 Grade 60 deformed bars with 90° hooks at both ends. All transverse reinforcement was #3 Grade 60 deformed bars with 135° seismic hooks at both ends, following Chapter 21 of ACI 318 (2008). Hoops were nominally spaced at \( d/4 \), or 2.5 in., with the first hoop at 1.25 in. from both stub faces. The transverse reinforcement ratio, \( \rho_t \), was 0.98%.

**B.4 Test Procedure**

Specimen CC-2.4-20 followed the same loading protocol as specimens UC-1.5-20 and UC-1.5-20F (described in Appendix A), refer to Figure B.3 and Table B.7. The specimen was loaded through the central stub in repeating cycles of step-wise increasing deformation amplitudes, reaching each amplitude twice. At the completion of the loading protocol, the specimen was pushed monotonically to failure or maximum drift amplitude attainable by the test apparatus.

**B.4.1 Test Setup**

Specimen CC-2.4-20 was tested in a horizontal loading system that used a steel block connected to a strong floor as the reaction force for the transverse load, and a strong wall as the reaction force for the roller supports. The hydraulic actuator applied transverse load through the stub using rods inserted into 1-in. conduit sleeves in the stub, as detailed in Figure B.2. The connection of the actuator to the specimen contained a swivel to allow translations and rotations of the specimen without inducing out-of-plane loads. Axial load was controlled by an external post-tensioning system consisting of four 1-in. diameter threaded rods tensioned with hydraulic rams. The rams were controlled manually via a manifold and hand pump, and applied equal force at all times during the test. Axial load
was measured in two of the four bars, then summed and doubled to attain the final monitored axial load. A schematic plan and elevation of the loading frame can be found in Figure B.4. Additional information on the details of the test setup can be found in sections A.4 and A.5 of Rautenberg (2011).

B.4.2 Measurements

Specimen displacements, strains in longitudinal reinforcing bars, concrete surface strains, and applied loads (lateral and axial) were measured. Displacements were measured using 8 LVDTs, positioned as shown in Figure B.5. Strains in the longitudinal reinforcing bars were measured using 10 strain gages, rated at 350 Ω. Figure B.6 shows the locations of strain gages. Concrete surface strains were measured using Whittemore gage instruments that consisted of an LVDT in an aluminum housing, similar to the instrument shown in Figure A.21. The gages measured the differential movements between the steel discs glued to the concrete surface as shown in Figure B.7. All instrumentation was connected to a National Instruments data acquisition system, consisting of a series of modules and terminal blocks that were coupled to SCXI chassis and connected to a computer. Additional details about the instrumentation can be found in section A.6 of Rautenberg (2011).

B.5 Measured Response

Specimen CC-2.4-20 was subjected to the same loading protocol as specimens UC-1.5-20 and UC-1.5-20F, described in Appendix A. The longitudinal reinforcing bars buckled during the final push at a drift ratio of 8%, causing a significant loss of transverse load carrying capacity. The yield point was identified by the moment when the longitudinal reinforcement first reached its yield strain, and was found to be at a shear force of 41 kips and at a drift ratio of 0.9%. The peak shear force was 41.6 kips, or $6.4\sqrt{f\'c[\text{psi}]}b \cdot d$. The limiting drift ratio was 8.0%, as dictated by the conditions listed in Chapter 3 of Rautenberg (2011). These values are tabulated in Table B.8. Figure B.8 shows the specimen at various points during the loading protocol, and Figure B.9 shows the specimen at the end of the test.
B.5.1 Measured Shear vs. Drift Ratio

Figures B.10 and B.11 show the Shear vs. Drift Ratio curves for specimen CC-2.4-20. Figure B.10 shows the south column, while Figure B.11 shows the north column, which controlled the loading protocol for this specimen. See section 3.2 of Rautenberg (2011) for additional information.

B.5.2 Measured Strains in Longitudinal Reinforcement

Figure B.12 shows the Strain vs. Drift Ratio for the 10 longitudinal reinforcing bar strain gages placed at locations indicated in Figure B.6. For the north column (controlling column), strain data were not fully available after a drift ratio of 2%. The limit on available data was due to the eventual loss of functionality in strain gages located at the face of the stub.

Rautenberg (2011) observed that the measured strains in the longitudinal reinforcement at the center of the stub were approximately 90% of the strains measured at the face of the stub, which could have led to large column rotations caused by slip of the reinforcement within the stub. He also observed that inelastic strains were greater in tension than compression, causing the tensile strains in the longitudinal reinforcement to compound and create a “ratcheting” effect in the strain vs. drift ratio curves (see TNWF in Figure B.12). Additional commentary can be found in section 3.3 of Rautenberg (2011).

B.5.3 Curvature

Figure B.13 shows a plot of moment against curvature at the face of the specimen for both north (controlling) and south columns. The data in these plots exist only up to a drift ratio of 2% (controlled by the north column), or step 9 of the loading protocol. These plots were generated using Equation B.1:

\[ \phi = \frac{\varepsilon_t - \varepsilon_b}{8''} \]  

(B.1)
where $\phi$ is the curvature, and the 8" is the distance between the top longitudinal bar and the bottom longitudinal bar. The values of $\varepsilon_t$ and $\varepsilon_b$ correspond to the measured strain for the top and bottom longitudinal reinforcing bars (a positive strain denotes elongation).

Additionally, mean curvature distributions were found using the differential displacements measured with the Whittemore gages. These curvature distributions are shown in Figures B.14 and B.15. The topmost line represents the largest drift ratio for which there is curvature distribution data. Figure B.14 shows the curvature distribution at the first peak of the first cycle for steps 1 to 9 of the loading protocol while Figure B.15 shows the curvature distribution at the second peak of the first cycle for steps 1 to 9 of the loading protocol. Additional details on how the curvature distribution was calculated can be found in section 3.4 of Rautenberg (2011).

B.5.4 Shear Deformations

The measured drift ratios are compared with the drift ratios derived from Whittemore data in Figure B.16. Good agreement between the measured and computed drift ratios was obtained by Rautenberg (2011) for specimen CC-2.4-20. Figure B.17 shows the fraction of the total drift that is associated with shear distortions for the north column of specimen CC-2.4-20. The shear component of drift was calculated using Whittemore gage data. The data suggest that at drift ratios below yielding, shear distortions account for approximately 10% of the total drift. At larger drift ratios, shear distortions account for nearly 15% of the total drift. Additional details on calculations and assumptions can be found in section 3.8 of Rautenberg (2011).
Appendix Tables
Table A.1 – Concrete Properties per Test Specimen

<table>
<thead>
<tr>
<th>Specimen ¹</th>
<th>Cast Date</th>
<th>Test Date</th>
<th>Compressive Strength² at Test Date psi</th>
<th>Modulus of Rupture³ at Test Date (f′/√f′c [psi])</th>
</tr>
</thead>
<tbody>
<tr>
<td>UC-1.5-20</td>
<td>6/23/2011</td>
<td>8/7/2012</td>
<td>4790</td>
<td>10</td>
</tr>
<tr>
<td>UC-1.5-20F</td>
<td>6/30/2011</td>
<td>12/13/2011</td>
<td>6600</td>
<td>22</td>
</tr>
</tbody>
</table>

¹ Specimen designation is based on the following:
UC: SAS 690 Gr. 97 longitudinal and ASTM A615 Gr. 60 transverse reinforcement
1.5: longitudinal reinforcement ratio of 1.5%; based on the area of longitudinal reinforcement, Aₚ, divided by the area of the gross section, Aₛ.
20: P/f′c ⋅ Aₛ = 20%, where P is axial load applied to the specimen
F: high performance fiber reinforced concrete (HPFRC) specimen with hooked steel fibers (Dramix RC-80/30-BP by Bekaert) in a fiber volume fraction of 1.5%
² Based on ASTM C39
³ Based on ASTM C78

Table A.2 – Concrete Mix Proportions (for 1 cubic yard)

<table>
<thead>
<tr>
<th>Item</th>
<th>Units</th>
<th>Target¹</th>
<th>UC-1.5-20</th>
<th>UC-1.5-20F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>lb.</td>
<td>700</td>
<td>404</td>
<td>912</td>
</tr>
<tr>
<td>Sand</td>
<td>lb.</td>
<td>1680</td>
<td>1380</td>
<td>1820</td>
</tr>
<tr>
<td>Gravel</td>
<td>lb.</td>
<td>1300</td>
<td>1760²</td>
<td>1280³</td>
</tr>
<tr>
<td>Water</td>
<td>lb.</td>
<td>280</td>
<td>183</td>
<td>183</td>
</tr>
<tr>
<td>Glenium</td>
<td>oz.</td>
<td>-</td>
<td>40</td>
<td>48</td>
</tr>
<tr>
<td>Air</td>
<td>%</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Fiber</td>
<td>Volumetric %</td>
<td>0 or 1.5</td>
<td>0</td>
<td>1.5</td>
</tr>
<tr>
<td>Water/Cement</td>
<td>-</td>
<td>0.4</td>
<td>0.37</td>
<td>0.29</td>
</tr>
</tbody>
</table>

¹ The 28-day compressive strength of concrete was specified as 6000 psi
² Maximum aggregate size of 1 in.
³ Maximum aggregate size of ½ in.
Table A.3 – Measured Steel Properties

<table>
<thead>
<tr>
<th>Reinforcing Bar/Size</th>
<th>Specimen</th>
<th>Coupon</th>
<th>$f_y$ ksi</th>
<th>$\varepsilon_{su}$</th>
<th>$f_u$ ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>#3 (Gr. 60)$^3$</td>
<td>All</td>
<td>-$^6$</td>
<td>66.9</td>
<td>0.138</td>
<td>104.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>100</td>
<td>0.109</td>
<td>118</td>
</tr>
<tr>
<td>#6 (Gr. 97)$^7,8$</td>
<td>All</td>
<td>$^2$</td>
<td>96</td>
<td>0.102</td>
<td>116</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$^3$</td>
<td>96</td>
<td>0.102</td>
<td>116</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td></td>
<td>97.3</td>
<td>0.104</td>
<td>116.7</td>
</tr>
</tbody>
</table>

$^1$ Based on ASTM A370.
$^2$ Defined using the 0.2%-offset method.
$^3$ Fracture elongation in 8-in. gage length.
$^4$ Tensile strength.
$^5$ Bars comply with ASTM A706 Grade 60.
$^6$ Data based on mill certificate.
$^7$ Threaded bars provided by SAS Stressteel. Bars comply with ICC-ES AC 237 (2009). Actual bar diameter is 0.70 in. (18 mm) with an area of 0.39 in$^2$ (250 mm$^2$).
$^8$ Measured steel properties reported by Tavallali (2011).
Table A.4 – Global As-Built Dimensions

<table>
<thead>
<tr>
<th>Dimension&lt;sup&gt;1&lt;/sup&gt;</th>
<th>Specimen</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UC-1.5-20</td>
</tr>
<tr>
<td>a</td>
<td>12</td>
</tr>
<tr>
<td>b</td>
<td>$36 \frac{1}{16}$</td>
</tr>
<tr>
<td>c</td>
<td>$16 \frac{1}{8}$</td>
</tr>
<tr>
<td>d</td>
<td>$36 \frac{1}{16}$</td>
</tr>
<tr>
<td>e</td>
<td>12</td>
</tr>
<tr>
<td>f</td>
<td>36</td>
</tr>
<tr>
<td>g</td>
<td>$16 \frac{1}{8}$</td>
</tr>
<tr>
<td>h</td>
<td>$36 \frac{1}{16}$</td>
</tr>
<tr>
<td>i</td>
<td>$5 \frac{1}{16}$</td>
</tr>
<tr>
<td>j</td>
<td>5</td>
</tr>
<tr>
<td>k</td>
<td>$5 \frac{1}{16}$</td>
</tr>
<tr>
<td>l</td>
<td>5</td>
</tr>
</tbody>
</table>

<sup>1</sup> All dimensions in inches. See Figure A.8 for dimension locations.
# Table A.5 – Cross-Section As-Built Dimensions

<table>
<thead>
<tr>
<th>Dimension(^1)</th>
<th>Specimen</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UC-1.5-20</td>
<td>UC-1.5-20F</td>
<td></td>
</tr>
<tr>
<td>Dimensions for North Column</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TVC</td>
<td>2</td>
<td>1(\frac{11}{16})</td>
<td></td>
</tr>
<tr>
<td>BVC</td>
<td>(1\frac{15}{16})</td>
<td>(1\frac{11}{16})</td>
<td></td>
</tr>
<tr>
<td>m</td>
<td>9</td>
<td>9(\frac{1}{16})</td>
<td></td>
</tr>
<tr>
<td>Dimensions for South Column</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TVC</td>
<td>(1\frac{3}{4})</td>
<td>(1\frac{11}{16})</td>
<td></td>
</tr>
<tr>
<td>BVC</td>
<td>(1\frac{3}{4})</td>
<td>(1\frac{11}{16})</td>
<td></td>
</tr>
<tr>
<td>m</td>
<td>9</td>
<td>9</td>
<td></td>
</tr>
</tbody>
</table>

\(^1\) All dimensions in inches. See Figure A.8 for dimension locations.
Table B.1 – Concrete Mix Proportions per Cubic Yard and Casting Data, Rautenberg (2011)

<table>
<thead>
<tr>
<th>Batch C03</th>
<th></th>
</tr>
</thead>
</table>
| Specimens | - \textit{CC-2.4-20}
| Date of Casting | - 8/7/09 |
| Curing Time | day 35 |
| Time Before Stripping Forms | day 4 |
| Cement | lb. 613 |
| #23 Sand | lb. 1280 |
| #8 Crushed Stone | lb. 1780 |
| Pea Gravel | lb. 0 |
| Water | lb. 230 |
| High-Range Water Reducer | oz. 12.5 |
| Steel Fibers | lb. 0 |
| Steel Fiber Type | - N/A |
| Fiber Length / Diameter | - N/A |
| Water-to-Cement Ratio | - 0.38 |
| Measured Slump | in. 4.0 |
Table B.2 – Concrete Strength and Modulus of Rupture on Test Day, Rautenberg (2011)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Age days</th>
<th>Batch</th>
<th>Compressive Strength psi</th>
<th>Modulus of Rupture psi</th>
<th>$\sqrt{f'_c}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>CC-2.4-20</td>
<td>44</td>
<td>C03</td>
<td>5300</td>
<td>5170</td>
<td>5280</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>550</td>
<td>575</td>
<td>7.9</td>
</tr>
</tbody>
</table>

Table B.3 – Concrete Compressive Strength Progression through 28 Days, Rautenberg (2011)

<table>
<thead>
<tr>
<th>Age days</th>
<th>Batch C03 psi</th>
<th>Ea.</th>
<th>Avg.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>3520</td>
<td>3500</td>
<td>3500</td>
</tr>
<tr>
<td></td>
<td>3540</td>
<td>3400</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>4060</td>
<td>3710</td>
<td>3850</td>
</tr>
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<td></td>
<td>3810</td>
<td></td>
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<td>14</td>
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<td>4200</td>
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<td>21</td>
<td>4520</td>
<td>4420</td>
<td>4500</td>
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<td></td>
<td>4500</td>
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<td>28</td>
<td>4590</td>
<td>4790</td>
<td>4700</td>
</tr>
<tr>
<td></td>
<td>4710</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table B.4 – Schedule of Longitudinal and Transverse Reinforcement, Rautenberg (2011)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Longitudinal Steel Heat</th>
<th>Longitudinal Steel Yield Stress ksi</th>
<th>Transverse Steel Heat</th>
<th>Transverse Steel Yield Stress ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>CC-2.4-20</td>
<td>S08</td>
<td>65</td>
<td>S11</td>
<td>62</td>
</tr>
</tbody>
</table>

Table B.5 – Reinforcement Sample Test Results and Averages, Rautenberg (2011)

<table>
<thead>
<tr>
<th>Steel Heat</th>
<th>Governing Specification</th>
<th>Bar Size in.</th>
<th>Yield Stress ksi</th>
<th>Ultimate Stress ksi</th>
<th>Elongation in 8 inches %</th>
</tr>
</thead>
<tbody>
<tr>
<td>S08</td>
<td>ASTM A706 Gr. 60</td>
<td>6/8</td>
<td>64 65 65</td>
<td>92 91 91</td>
<td>15.6 15.6 18.0</td>
</tr>
<tr>
<td>S11</td>
<td>ASTM A615 Gr. 60</td>
<td>3/8</td>
<td>62 61 62</td>
<td>92 90 94</td>
<td>12.5 15.6 14.3</td>
</tr>
</tbody>
</table>
Table B.6 – Selected Properties of Specimen CC-2.4-20, Rautenberg (2011)

| Designation | Longitudinal Reinforcement | Transverse Reinforcement | \( P \) | \( f'_c \) | \( P \) |
|-------------|-----------------------------|--------------------------|---------|---------|
| CC-2.4-20   | \( d_b \) 6 in. 6/8 2.4   | \( s \) 2.5 in. 1.0     | 62      | 19      | 5250  |
|             | \( \rho_g \% \) 65        | \( \rho_v \% \) 62       |         |         | 108   |

\( d_b \) : Diameter of longitudinal reinforcing bar

\( \rho_g \) : Longitudinal reinforcement ratio based on gross cross-sectional area

\( f_y \) : Measured yield stress of longitudinal reinforcement

\( s \) : Hoop spacing

\( \rho_v \) : Transverse reinforcement ratio

\( f_{yy} \) : Measured yield stress of transverse reinforcement

\( P \) : Applied axial load

\( f'_c \) : Concrete compressive strength

\( A_g \) : Gross cross-sectional area of column

Table B.7 – List of Target Drift Ratios Used in Loading Protocol, Rautenberg (2011)

<table>
<thead>
<tr>
<th>Drift Target</th>
<th>Target Drift Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.15</td>
</tr>
<tr>
<td>2</td>
<td>0.2</td>
</tr>
<tr>
<td>3</td>
<td>0.3</td>
</tr>
<tr>
<td>4</td>
<td>0.4</td>
</tr>
<tr>
<td>5</td>
<td>0.6</td>
</tr>
<tr>
<td>6</td>
<td>0.8</td>
</tr>
<tr>
<td>7</td>
<td>1.0</td>
</tr>
<tr>
<td>8</td>
<td>1.5</td>
</tr>
<tr>
<td>9</td>
<td>2.0</td>
</tr>
<tr>
<td>10</td>
<td>3.0</td>
</tr>
<tr>
<td>11</td>
<td>4.0</td>
</tr>
<tr>
<td>12</td>
<td>5.0</td>
</tr>
</tbody>
</table>

\(^1\) The loading protocol follows the recommendations of FEMA 461 (2007). It consists of 12 steps (drift targets), two cycles were applied at each step.
Table B.8 – Measured Yield and Ultimate Shear Forces and Drift Ratios for Specimen CC-2.4-20, Rautenberg (2011)

<table>
<thead>
<tr>
<th>Designation</th>
<th>$V_y^1$</th>
<th>$\theta_y^1$</th>
<th>$V_u^2$</th>
<th>$\nu_u^2$</th>
<th>$\theta_u^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>CC-2.4-20</td>
<td>41.0</td>
<td>0.9</td>
<td>41.6</td>
<td>6.4</td>
<td>8.0</td>
</tr>
</tbody>
</table>

1 Shear and drift ratio at which the strain in the longitudinal reinforcement first reaches its yield strain (based on 0.2% offset method).

2 Maximum shear, $V_u$, and maximum shear stress, $\nu_u$, based on $V_u/(bd)$.

3 Drift capacity, $\theta_u$, for specimen CC-2.4-20 was controlled by bar buckling during the monotonic loading (final push). At a drift ratio of 8%, the specimen strength dropped below 80% of the peak load that corresponds to step 12 (5% drift ratio) of the loading protocol.
Appendix Figures
Figure A.1—Measured Compressive Strength of Concrete vs. Time, Specimen UC-1.5-20

Figure A.2—Measured Compressive Strength of Concrete vs. Time, Specimen UC-1.5-20F
Figure A.3—Compressive Tests of 4x8" Concrete Cylinders
Figure A.4—Flexural Tests of 4x4x14” Concrete Beams
Figure A.5—Defining Yield Strength of #6 (Gr. 97) Bars using 0.2%-Offset Method, Tavallali (2011)

Figure A.6—View of Loading Frame and Test Specimen (Dimensions in Inches)
Figure A.7—View of Axial Loading Frame and Test Specimen (Dimensions in Inches)

(2) 1-1/2" thick plates compressing the specimen with axial load

(2) Roller end-plates to transfer axial load to center of specimen cross-section

(4) 1-1/2" Ø bars
\[ f_y = 125 \text{ ksi} \]

(4) Powerteam center-hole hydraulic cylinders, model RH303 to apply tension to bars (cylinders push in direction of arrows)

(4) Hex nuts and 1/4" thick plates to function as reaction surface for cylinders

Figure A.8—Dimension Locations for Table A.4 and Table A.5
Figure A.9—Details of Steel Reinforcement Used in All Columns

Figure A.10—Details of Formwork (Dimensions in Feet and Inches)
Figure A.11—Assembled Wood Formwork
Figure A.12—Assembled Steel Cages
Figure A.13—Pouring Concrete for Specimen UC-1.5-20
Figure A.14—Finishing of Specimen UC-1.5-20
Figure A.15—Finishing of Specimen UC-1.5-20F

Figure A.16—Loading Frame with Specimen, Partially Assembled
Figure A.17—Specimen with Endplate and Axial Post-Tensioning Bars, Partially Assembled
Figure A.18—Component Details of Axial Post-Tensioning System (Dimensions in Inches)
Figure A.19—Components Used in Calibration

a) Overall Calibration Setup

b) Cylinder, Reaction Plate, and Nut

c) Reference MTS 661.23A-02 Load Cell
Drift = Δ/a
Adjusted South Drift = (Δ+ωβ)/a+β.
Adjusted North Drift = (Δ-ωβ)/a-β

Figure A.20—Calculation of Drift

Figure A.21—Whittemore Gage Instrument, Pfund (2012)
Figure B.1—Nominal Dimensions of Specimen CC-2.4-20, Rautenberg (2011)

Figure B.2—Specimen CC-2.4-20 Dimensions and Reinforcement Layout, Rautenberg (2011)
Figure B.3—Specimen CC-2.4-20 Displacement History, Rautenberg (2011)
Figure B.4—Typical Test Setup, Rautenberg (2011)
a) Plan View

b) Elevation View

Figure B.5—Specimen CC-2.4-20 LVDT Layout, Rautenberg (2011)
Figure B.6—Specimen CC-2.4-20 Strain Gage Layout in Plan View, Rautenberg (2011)

Figure B.7—Specimen CC-2.4-20 Whittemore Gage Point Layout, Rautenberg (2011)
Figure B.8—Specimen CC-2.4-20 during the Test, Rautenberg (2011)
Figure B.9—Specimen CC-2.4-20 at the End of the Test, Rautenberg (2011)
Figure B.10—Shear vs. Drift Ratio for Specimen CC-2.4-20, South Column, Rautenberg (2011)

Figure B.11—Shear vs. Drift Ratio for Specimen CC-2.4-20, North Column, Rautenberg (2011)
Figure B.12—Measured Longitudinal Strain vs. Drift Ratio for Specimen CC-2.4-20, Rautenberg (2011)
Figure B.12—Continued
Figure B.13—Measured Moment vs. Curvature at the Stub Face of Specimen CC-2.4-20
Figure B.14—Curvature Distribution for Positive Drift Ratios in Specimen CC-2.4-20
Derived from Whittemore Data, Rautenberg (2011)

Figure B.15—Curvature Distribution for Negative Drift Ratios in Specimen CC-2.4-20
Derived from Whittemore Data, Rautenberg (2011)
Figure B.16—Measured Drift vs. Drift Derived from Whittemore Data for Specimen CC-2.4-20, Rautenberg (2011)

Figure B.17—Measured Component of Shear Drift vs. Total Drift for Specimen CC-2.4-20
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Biographical Sketch

Ksenia Tretiakova

Ksenia Tretiakova was born on July 31 in Akademgorodok, Russia, just outside the capital city of Siberia, Novosibirsk. At age 4, she and her mother moved to the United States, and took residence in Philadelphia, where she attended McCall School and Julia Reynolds Masterman Laboratory and Demonstration School. In fall of 2007, she began the Bachelor of Architectural Engineering (BAE) program at The Pennsylvania State University and was accepted to the Schreyer Honors College. In spring of 2011, she began the integrated BAE/MS program under Dr. Andres Lepage. She expects to graduate in August of 2013.

During the summer months of 2007, Ksenia worked as an intern at Intrexon Corporation in Blacksburg, VA. Ksenia spent the following summers of 2008, 2009, and 2010 working in the Building Envelope Research Laboratory at Penn State under Paul Kremer, assisting in research concerning the performance of various glazing under earthquake-induced forces and the effectiveness of various polymers in the design of brick facades for blast-resistance.