Rehabilitation of non-ductile reinforced concrete building columns using fiber reinforced polymer jackets

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Rehabilitation of Non-Ductile Reinforced Concrete Building Columns
Using Fiber Reinforced Polymer Jackets

by

Stephanie L. Walkup

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ABSTRACT

Many reinforced concrete frame structures built prior to the 1970's were designed for gravity and wind loads. Reinforcing details used in the columns of these structures are associated with four non-ductile failure modes under seismic loading: axial flexural, ductile shear, brittle shear, and lap splice failure. This investigation focuses on the axial flexural and ductile shear failure modes. Fiber reinforced polymer (FRP) jackets are a recent retrofit method for non-ductile reinforced concrete columns. The research objectives are to investigate FRP jackets as a retrofit against axial flexural and ductile shear failures, and to evaluate analytical methods and design guidelines for FRP jackets.

Seven full-scale building column specimens were tested under axial and cyclic lateral load. One non-retrofit specimen failed in the axial flexural mode, and one non-retrofit specimen failed in the ductile shear mode. Three specimens were retrofit against axial flexural failure, and two against ductile shear failure. The specimens were retrofit with carbon FRP jackets. The jacket strength and stiffness were controlled by the number of plies of FRP material. The jackets were designed to increase ductility without increasing flexural strength or stiffness.

The findings are: (1) the non-retrofit specimens failed as expected; (2) the ductility of the retrofit specimens was limited by jacket rupture or by rupture of the longitudinal reinforcing bars; (3) the behavior of the ductile shear specimens was significantly improved by the jackets and the ductile shear failure mode was eliminated; (4) the non-retrofit and retrofit specimens had the same flexural strength and stiffness; (5) the jacket properties did not influence the energy dissipation per cycle of the retrofit specimens and failure was not preceded by deterioration in hysteretic behavior; (6) the displacement ductility capacity of the retrofit specimens was significantly greater than that of the non-retrofit specimens and increased with the number of jacket plies.
The conclusions are: (1) FRP jacket retrofit significantly increases the ductility capacity of non-ductile reinforced concrete building columns; (2) the controlling limit states columns with FRP jackets are jacket rupture and reinforcing bar rupture; (3) for a well-designed jacket the controlling limit state is reinforcing bar rupture; (4) existing FRP jacket design guidelines for bridge pier columns provide conservative jacket designs for building columns similar to those considered herein, and jackets with less material performed nearly as well.
CHAPTER 1
INTRODUCTION

1.1 INTRODUCTION

Many reinforced concrete frame structures built in North America prior to the 1970's were designed for either gravity loads alone, or gravity loads and wind loads. Seismic loads often were not treated in the design of these structures. Many of the reinforcing details used in these structures are now recognized to be associated with non-ductile failure modes under seismic loading. As a result, poor performance of these structures is anticipated under moderate to severe seismic loading.

As a result of poor reinforcing details, and lack of consideration of seismic loads in the original design, the columns are often found to be deficient in these non-ductile reinforced concrete structures. The columns have four potential failure modes: axial flexural failure, ductile shear failure, brittle shear failure, and lap splice failure. Each of these failure modes is discussed in Chapter 2. Current work at Lehigh University addresses retrofit for against these four failure modes. This investigation focuses on retrofit against axial flexural and ductile shear failures.

Jacketing is a method often used to retrofit reinforced concrete columns. Columns may be jacketed with additional reinforced concrete, steel, or various fiber reinforced polymer (FRP) materials. Jackets may be used to restore (in the case of damaged or deteriorated columns), maintain, or increase axial load capacity, flexural capacity, and/or shear capacity.

Steel and reinforced concrete jackets are often used to rehabilitate or retrofit damaged, deteriorated, or otherwise deficient columns. Recently, FRP jackets have emerged as a potential retrofit for concrete structures. The benefits of FRP jackets include high strength-to-weight and stiffness-to-weight ratios, corrosion resistance, and controlled anisotropy (Argarwal and Broutman,
The ability to control the properties of FRP materials in specific directions allows FRP jackets to be tailored to the needs of each column retrofit.

1.2 OBJECTIVES

Previous research has been conducted to evaluate the use of FRP jackets as a means to retrofit reinforced concrete members. Much of this research has consisted of axial load tests of reinforced concrete columns, and combined axial and lateral load tests of reinforced concrete bridge pier columns. However, the use of FRP jackets to retrofit non-ductile reinforced concrete building columns under combined axial and lateral loads needs to be investigated.

The objectives of this investigation are as follows:

(1) To investigate the use of FRP jackets as a retrofit against axial flexural and ductile shear failures in non-ductile reinforced concrete building columns.

(2) To evaluate analytical methods and design guidelines for the retrofit of non-ductile reinforced concrete building columns using FRP jackets.

1.3 SUMMARY OF APPROACH

Seven full-scale square building column specimens with the same dimensions and reinforcing details were tested. Two non-retrofit specimens were tested as control specimens. The first non-retrofit specimen was tested to fail in the axial flexural mode. The second non-retrofit specimen was tested to fail in the ductile shear mode. The remaining five specimens included three specimens retrofit against the axial flexural failure mode, and two specimens retrofit against the ductile shear failure mode. The column specimens were retrofit with carbon FRP jackets. The primary variables treated in the retrofit specimens are the strength and stiffness of the jackets which are controlled by the number of plies (layers) of carbon FRP material used in the jackets, and the expected failure mode if the specimens were not retrofit. Details of the test specimens and test procedure are given in Chapter 4.
1.4 SCOPE OF REPORT

Chapter 2 provides background information relevant to this report. A brief review of typical details in non-ductile reinforced concrete frame structures is presented, and the four potential failure modes of non-ductile columns are discussed. The use of jackets as a retrofit method is explained, and previous research on FRP jackets is summarized. Confined concrete models are reviewed. Previous tests of jacketed concrete columns are reviewed.

Chapter 3 reviews the current design guidelines for the retrofit of non-ductile reinforced concrete bridge pier columns using FRP jackets. Chapter 4 provides details of the experimental program, and a description of the carbon FRP material used in this study. Analyses of non-retrofit and retrofit specimens are presented. The designs of the carbon FRP jackets for the retrofit specimens are discussed. Procedures used to apply the carbon FRP jackets to the test specimens are also discussed. Results of tests of the non-retrofit and retrofit specimens are presented in Chapter 5.

Chapter 6 discusses the results of the tests. The strength, stiffness, ductility, and energy dissipation characteristics of the test specimens are presented and discussed. The local behavior of the test specimens in the form of curvatures and jacket strain data is presented. The results of the analyses discussed in Chapter 4 are compared with the experimental test results. Analytical results for displacement ductility, curvatures and extreme fiber concrete compressive strains are evaluated, based on test results.

Chapter 7 summarizes the study and presents the conclusions of the research. Suggestions for further research are also given in this chapter.

1.5 NOTATION

The following notation is used in this report:

\[ a = \text{rectangular stress block depth, mm;} \]
\( A_c \) = area of concrete, \( \text{mm}^2 \);
\( A_{ce} \) = area of concrete confined within the transverse reinforcement, \( \text{mm}^2 \);
\( A_e \) = effective area of concrete, \( \text{mm}^2 \);
\( A_g \) = gross area of the concrete cross-section, \( \text{mm}^2 \);
\( A_h \) = area of hoop or spiral reinforcement, \( \text{mm}^2 \);
\( A_t \) = area of transverse steel, \( \text{mm}^2 \);
\( b \) = width of the column in the direction perpendicular to loading, \( \text{mm} \);
\( b_w \) = width of the web of the column in the direction perpendicular to loading, \( \text{mm} \);
\( c \) = neutral axis depth of the column cross section, \( \text{mm} \);
\( c_u \) = neutral axis depth of the column cross section corresponding to \( \sigma_u \), \( \text{mm} \);
\( cc \) = concrete cover to the longitudinal reinforcement, \( \text{mm} \);
\( d \) = depth from the extreme concrete compressive fiber to the location of the tensile reinforcing bars, \( \text{mm} \);
\( d_b \) = diameter of reinforcing bar, \( \text{mm} \);
\( d_s \) = diameter of spiral, \( \text{mm} \);
\( D \) = column dimension in the loading direction, \( \text{mm} \);
\( D' \) = core column dimension in the loading direction, \( \text{mm} \);
\( D_d \) = diagonal dimension of a square column cross section, \( \text{mm} \);
\( D_j \) = diameter of the jacket, \( \text{mm} \);
\( E_{1/2\ \text{cycle}} \) = energy dissipation per half cycle, \( \text{MPa} \);
\( E_c \) = modulus of elasticity of concrete, \( \text{MPa} \);
\( E_t \) = FRP material stiffness per unit dimension perpendicular to the fiber orientation per ply, \( \text{kN/mm/ply} \);
\( E_j \) = modulus of elasticity of jacket, \( \text{MPa} \);
$E_j = \text{in situ FRP material stiffness per unit dimension perpendicular to the fiber orientation per ply, kN/mm/ply;}$

$E_s = \text{modulus of elasticity of steel, MPa;}$

$f_c = \text{compressive concrete stress, MPa;}$

$f_c' = \text{compressive strength of concrete, MPa;}$

$f_{co} = \text{compressive strength of unconfined concrete, MPa;}$

$f_{cc} = \text{compressive strength of confined concrete, MPa;}$

$f_{cc}' = \text{compressive strength of confined concrete, MPa;}$

$f_{con} = \text{confinement stress, MPa;}$

$f_b = \text{clamping force provided by the transverse reinforcement, MPa;}$

$f_j = \text{jacket stress, MPa;}$

$E_j = \text{in situ strength of FRP material per unit dimension perpendicular to the fiber orientation per ply, N/mm/ply;}$

$f_l = \text{lateral clamping pressure required to maintain lap splice capacity, MPa;}$

$f_{sl} = \text{steel stress in longitudinal reinforcement, MPa;}$

$f_{\text{residual}} = \text{residual concrete stress of unconfined concrete, MPa}$

$f_s = \text{steel stress, MPa;}$

$f_y = \text{yield stress of the longitudinal reinforcement; MPa;}$

$f_{sy} = \text{yield stress of the longitudinal reinforcement, MPa;}$

$f'_y = \text{yield stress of the transverse reinforcement, MPa;}$

$f_{sj} = \text{yield stress of the jacket material, MPa;}$

$h_1 = \text{height of applied lateral load, mm;}$

$h_2 = \text{height of the column, mm;}$

$h_3 = \text{height of the applied axial load, mm;}$
H = applied lateral load, kN;

$I_{kd}$ = moment of inertia of the column cross section at a given neutral axis depth, mm$^4$;

$I_{cl}$ = moment of inertia of the uncracked column cross section, mm$^4$;

$I_{ut}$ = moment of inertia of the cracked column cross section, mm$^4$;

$I_e$ = effective moment of inertia of the column cross section, mm$^4$;

$I_g$ = gross moment of inertia of the column cross section, mm$^4$;

$k$ = strength reduction factor based on $\mu_A$;

$kd$ = neutral axis depth, mm;

$K$ = stiffness of column,

$K_{el}$ = the elastic stiffness of column,

$L$ = clear column height, mm;

$L_{c1}$ = length of the primary confinement region for the flexural hinge, mm;

$L_{c2}$ = length of the secondary confinement region adjacent to the flexural hinge, mm;

$L_{v1}$ = length of region retrofit against shear failure inside the flexural hinge, mm;

$L_{v0}$ = length of the region retrofit against shear failure outside the flexural hinge, mm;

$L_p$ = length of the plastic hinge region, mm;

$L_s$ = length of the lap splice, mm;

$M_a$ = moment at an arbitrary lateral load, kN mm;

$M_{cr}$ = cracking moment of concrete cross section, kN mm;

$M_n$ = nominal moment capacity of concrete cross section, kN mm;

$M_p$ = plastic moment capacity, kN m;

$M_p$ = probable moment capacity, kN m;

$M_y$ = moment in the column corresponding to yielding of the column, kN mm;

$n$ = number of plies;
\( P \) = applied axial load, kN;
\( p \) = inside crack perimeter along the longitudinal column reinforcement, mm;
\( s \) = center to center spacing of transverse reinforcement, mm;
\( t_j \) = thickness of the jacket, mm;
\( T_j \) = tensile force in the jacket, kN
\( U_{cc} \) = strain energy of the core concrete confined by and contained within the transverse reinforcement per unit length of the column, kN;
\( U_{co} \) = strain energy capacity of the concrete contained within the transverse reinforcement per unit length of the column assuming the concrete is unconfined, kN;
\( U_{sh} \) = strain energy capacity of the transverse reinforcement per unit length of the column, kN;
\( U_{sc} \) = strain energy required to maintain yield in the longitudinal steel in compression per unit length of the column, kN;
\( U_j \) = strain energy capacity of the jacket per unit length of the column, kN;
\( V_c \) = concrete contribution to shear strength, kN;
\( V_c^i \) = concrete contribution to shear strength inside the plastic hinge region, kN;
\( V_c^o \) = concrete contribution to shear strength outside the plastic hinge region, kN;
\( V_j \) = jacket shear strength, kN;
\( V_o \) = required shear strength, kN;
\( V_n \) = nominal shear strength, kN;
\( V_p \) = axial load contribution to shear strength, kN;
\( V_{pr} \) = probable shear resistance, kN;
\( V_s \) = transverse steel contribution to shear strength, kN;
\( \gamma_1 \) = integration coefficient for concrete;
\( \gamma_2 \) = integration coefficient for steel;
\( \delta \) = displacement of column at point of lateral load application, mm;

\( \delta_y \) = displacement of column at point of lateral load application corresponding to yield of the column, mm;

\( \Delta_1 \) = displacement corresponding to \( h_1 \), mm;

\( \Delta_2 \) = displacement corresponding to \( h_2 \), mm;

\( \Delta_3 \) = displacement corresponding to \( h_3 \), mm;

\( \varepsilon_c \) = axial concrete strain, mm/mm;

\( \varepsilon_{c_0} \) = axial concrete strain corresponding to \( f_{c_0} \), mm/mm;

\( \varepsilon_{c_c} \) = axial concrete strain corresponding to \( f_{c_c} \), mm/mm;

\( \varepsilon_{e, ef} \) = extreme fiber concrete compression strain, mm/mm;

\( \varepsilon_{c, ef} \) = concrete compressive strain at extreme fiber, mm/mm;

\( \varepsilon_{e_m, ef} \) = maximum extreme fiber concrete compressive strain, mm/mm;

\( \varepsilon_{e_m, ef} \) = residual concrete strain at zero applied lateral load, mm/mm;

\( \varepsilon_{c, ef} \) = ultimate axial compressive strain of concrete, mm/mm;

\( \varepsilon_f \) = axial FRP material strain, mm/mm;

\( \varepsilon_{fr} \) = axial FRP material strain at rupture, mm/mm;

\( \varepsilon_j \) = \textit{in situ} axial FRP jacket strain, mm/mm;

\( \varepsilon_{pl} \) = limiting dilation strain of FRP jacket, mm/mm;

\( \varepsilon_{pr} \) = \textit{in situ} axial FRP jacket strain at rupture, mm/mm;

\( \varepsilon_{\text{residual}} \) = the residual concrete tensile strain as the applied lateral load approaches zero, mm/mm;

\( \varepsilon_{e_i} \) = the extreme fiber concrete compression strain increment, mm/mm;

\( \varepsilon_s \) = steel strain, mm/mm;

\( \varepsilon_{sf} \) = transverse reinforcement fracture strain, mm/mm;
\( \varepsilon_{sp} \) = axial strain corresponding to spalling of the concrete, mm/mm;

\( \eta \) = limiting dilation ratio;

\( \theta \) = angle of the principle compression strut, rad;

\( \mu \) = friction coefficient of concrete;

\( \mu_d \) = displacement ductility;

\( \mu_\theta \) = rotational ductility;

\( \mu_\phi \) = curvature ductility;

\( \rho_{cc} \) = volume ratio of the longitudinal reinforcement to the core concrete;

\( \rho_j \) = volume ratio of the jacket to the confined concrete;

\( \rho_s \) = volume ratio of the longitudinal reinforcing steel to the concrete;

\( \rho_{sj} \) = volume ratio of the steel jacket to the confined concrete;

\( \rho_t \) = volume ratio of the transverse steel to the concrete;

\( \phi \) = curvature of the column cross section, rad/mm;

\( \phi_u \) = ultimate curvature of the column cross section corresponding to \( \varepsilon_{cu} \), rad/mm;

\( \phi_y \) = yield curvature of the column cross section corresponding to yielding of the column, rad/mm;

1.6 CONVERSION FACTORS

The unit conversion factors used in this report are given below. Additionally, reinforcing bar sizes are given the designation appropriate to their country of origin followed by their actual diameters. In the text, a bar designated with a "#" sign (e.g.: #3) refers to the standard inch-pound designation used in the United States (the number refers to the bar diameter in eighths of an inch). Standard metric bars (Canada, Europe and Japan) are designated with "No." (e.g.: No. 10). In this
case the number refers to the *nominal* bar diameter. Bar sizes have been reported in the same manner they were designated in the original references.

1 in = 25.4 mm

1 kip = 4.448 kN

1 ksi = 6.895 MPa

1 lb/ft$^3$ = 16.02 kg/m$^3$

1 ft-lb = 1.356 N-m
CHAPTER 2

BACKGROUND

This chapter presents the background information for this research. Section 2.1 identifies columns as critical non-ductile members. Section 2.2 describes the seismic behavior of non-ductile reinforced concrete columns. Section 2.3 describes the retrofit objectives of non-ductile reinforced concrete columns. Section 2.4 reviews confined concrete models. Section 2.5 discusses the retrofit of non-ductile reinforced concrete columns using steel and reinforced concrete jackets. Section 2.6 describes the retrofit of non-ductile reinforced concrete columns using FRP jackets.

2.1 IDENTIFICATION OF COLUMNS AS CRITICAL NON-DUCTILE MEMBERS

As noted in Chapter 1, reinforced concrete frame structures built prior to the 1970's in North America were designed for either gravity loads alone or wind and gravity loads only. Seismic loads were not considered in the design of these structures. Pessiki et al. (1990) and Wu (1995) conducted extensive reviews of design and detailing manuals in use during the period in which these structures were designed. This was done to assess the importance of the details in these structures when subject to seismic loading. Several reinforcing details which are potentially critical to the seismic behavior of reinforced concrete frame structures were identified (see Figure 2.1):

(1) Poorly confined columns with widely spaced transverse reinforcement;
(2) Poorly confined column lap splices located in the region of maximum moment;
(3) Discontinuous positive (bottom) beam flexural reinforcement embedded in the beam column joint;
(4) Beam columns joints with little or no transverse reinforcement; and,
(5) Construction joints above and below the beam-column joint.
Kurama et al. (1996) investigated the response of a series of non-ductile frame structures subject to seismic loading. Structures that included the details listed above were designed and analyzed to determine the importance of these details and the behavior of the structures. The parameters considered in the structures that were analyzed included the number of stories, the number of bays, and the design methodology (both Allowable Stress Design and Ultimate Strength Design were considered). The structures included detailing typical of the 1960's and were designed for gravity loads only. One of the primary failure modes found to occur in the structures that were studied was axial flexural compression failure in columns, which is associated with a non-ductile, weak-column-strong beam failure (Kurama et al. 1996). This failure mode is a result of the lack of adequate confinement of the concrete, which is caused by widely spaced transverse reinforcement.

Additional analyses were performed of retrofit structures that contained various amounts of strength and/or ductility increases to selected key regions in the structures. In these analyses, Kurama et al. found that improvements in column ductility without changes in column strength and stiffness, improved the overall behavior of the structures. For this reason, column retrofit may prove a suitable technique for improving overall building response.

Columns may be retrofit in several ways. Jacketing the concrete column to increase column ductility was the recommended form of retrofit. Jacket retrofits can be used to increase column strength, stiffness, and/or ductility in selected regions, preventing undesirable non-ductile failure (Wu 1995). In addition to determining the retrofit technique, column curvature ductility values and overall structural displacement ductility values, corresponding to different drift limits were developed for the prototype structures (see Table 2.1). This previous research by Kurama et al. and Wu provides the motivation to focus on the retrofit of reinforced concrete building columns as a means to retrofit the entire structure.
2.2 SEISMIC BEHAVIOR OF NON-DUCTILE REINFORCED CONCRETE COLUMNS

2.2.1 Typical Details of Non-Ductile Reinforced Concrete Building Columns

The current and all previous editions of the ACI 318 Code (ACI 318-1941, 1947, 1951, 1956, 1963, 1971, 1977, 1983, 1989, and 1995) were reviewed to study the code provisions that pertain to the transverse reinforcing and lap splice details in reinforced concrete columns. The results of this review are summarized in Table 2.2. Included in this table are provisions relating to general requirements for columns (minimum tie size, maximum tie spacing, requirements for shear design, minimum shear reinforcement, maximum spacing of shear reinforcement, tie configuration requirements, and minimum column bar lap splice lengths). Also shown in Table 2.2 are special provisions for seismic design which first appeared in the 1971 Code (length of the assumed plastic hinge region, maximum tie spacing requirements over the length of the column, minimum shear reinforcement, special anchorage details, and minimum lap splice length).

Many provisions summarized in Table 2.2 treat requirements for the transverse reinforcement in columns. These are key provisions that influence the behavior of the columns. In plastic hinge zones, the transverse reinforcement serves to confine the concrete, which helps to maintain the compressive resistance of the concrete under large compressive strains and also to maintain the shear resistance of the concrete. Also, transverse reinforcement provided throughout a column provides an additional strength contribution to shear resistance. Finally, transverse reinforcement provides support against longitudinal steel bar buckling.

Many earlier provisions summarized in Table 2.2 result in column details that contribute to possible non-ductile behavior under seismic loading. The implications of several of these key details are discussed below.

Smaller Diameter of Transverse Reinforcement - Transverse reinforcement with a small diameter is less effective in resisting buckling of longitudinal reinforcing bars and providing
confinement to the concrete core. As a result, small diameter transverse reinforcement results in a decrease in available ductility. As shown in Table 2.2, #3 and #4 bars are current minimum bar sizes permitted for column ties. Prior to 1971, designers were permitted to specify #2 bars.

**Tie Spacing** - Although general requirements for tie spacing have not changed, seismic requirements, first introduced in the 1971 Code, although relegated to an Appendix until 1989, have become more restrictive. Requirements for smaller tie spacing provide increased confinement to the concrete core and improved restraint against the buckling of longitudinal reinforcing bars.

**Tie Anchorage** - Typically, ties are only required to have 90 degree anchorage bends. Such an anchorage provides little confinement once the concrete has spalled, since the anchorage easily straightens, allowing the tie to come away from the concrete core. In such a case, the tie no longer provides confinement to the concrete core nor buckling restraint to the longitudinal reinforcement. Seismic detailing provisions require 135 degree anchorages into the concrete core for all ties in the plastic hinge region. These ties are referred to as “seismic hoops”. Using this detailing, anchorage of the tie and thus confinement is not lost when the concrete cover spalls.

**Lack of Provisions for Shear** - Specific provisions for transverse reinforcing steel to carry shear forces were introduced in ACI 318-71. Until this time, minimum tie reinforcements were typically used. After 1971, transverse steel was required for cases where the shear in the column exceeded one half of the shear force that may be carried by the concrete alone. For this reason, the minimum shear requirements were increased, thus also increasing the amount of confinement provided to the concrete and for support of the longitudinal reinforcing bars. Reviewing the seismic portion of the ACI code, it is found that the concrete contribution to shear resistance is related to the confined concrete core, but it is typically neglected. This provision would result in a higher shear requirement and ultimately better confinement of the concrete core.
Lap Splice Length and Location - Provisions for minimum lap splice lengths were first introduced ACI 318-56. Until this time, no minimum length was required. Provisions for lap splice location did not arise in the ACI 318 until 1971. Prior to this, bars were spliced together at convenient construction locations, often just above a floor level, where the column moments may be expected to be greatest. Thus three factors contribute to potentially poor seismic performance of a column splice: splice location, splice length, and inadequate confinement along the splice.

2.2.2 Potential Failure Modes of Non-Ductile Reinforced Concrete Columns

The typical column details described in the previous section suggest four possible non-ductile failure modes: axial flexural failure, ductile shear failure, brittle shear failure, and lap splice failure. Each of these failure modes is discussed below.

Axial flexural failure - Columns constructed with details described above and which include continuous longitudinal reinforcement at the ends of the column may be able to develop flexural plastic hinges at the ends under seismic loading. However, the small amount of transverse reinforcement may result in inadequate concrete confinement and inadequate restraint of longitudinal bar buckling under ductility demands associated with seismic loading. As a result, the longitudinal bars may buckle and the concrete core may deteriorate under combined axial force and bending moment shortly after the concrete cover spalls. Both the flexural resistance and axial force resistance may be lost. This non-ductile behavior is defined as an axial flexural failure.

Ductile shear failure - In a ductile shear failure, the column will first yield in flexure in the hinge region, and with increasing ductility demand, will eventually fail in shear in the hinge region. The failure occurs because the shear strength in the plastic hinge region deteriorates with increases in flexural ductility demands. As plastic-hinge rotations increase, the flexural-shear cracks widen and the compressive strains in the concrete increase. Both of these actions reduce the shear resistance of the concrete. Widening of flexural cracks contributes to this reduction in shear
resistance because of a reduction in shear transfer by aggregate interlock. The shear resistance in the plastic hinge zone is often assumed to decrease linearly as the ductility demand increases (Priestley et al. 1994). A ductile shear failure occurs when the aggregate interlock mechanism of the concrete shear resistance decreases significantly. The column can no longer support the applied loads and fails in shear.

**Brittle shear failure** - In a brittle shear failure, the column fails in shear prior to yielding in flexure. This failure mode is often associated with shorter columns having smaller moment-to-shear ratios at their ends. This failure mode may also be associated with columns in which transverse reinforcement of the column hinge region is larger than in the mid-height region. As opposed to a ductile shear failure, a brittle shear failure occurs when the shear force is greater than the shear strength. The shear capacity, therefore, does not need to deteriorate to precipitate a brittle shear failure.

**Lap splice failure** - A lap splice failure is associated with failure of the concrete along the interface between the lapped bars and splitting of the concrete cover over the bars. When a lap splice failure occurs, the column is no longer able to develop its flexural capacity.

**2.3 RETROFIT OBJECTIVES FOR NON-DUCTILE REINFORCED CONCRETE COLUMNS**

Each of the failure modes described above needs to be considered when designing the retrofit for a non-ductile column. As noted in Chapter 1, this report focuses on the first two failure modes: axial-flexural failure, and ductile shear failure. Furthermore, FRP column jacketing is the specific retrofit technique treated in this research.

An FRP jacket may be designed to provide increased column ductility or increased column strength. Possible jacket design objectives may include one or a combination of the following:
(1) Increase concrete confinement and longitudinal bar restraint in order to enhance the ductility of an axial-flexural failure mode;

(2) Increase concrete confinement to maintain the shear strength of the flexural hinge region, thereby preventing a ductile shear failure;

(3) Increase the shear strength of the column to prevent a shear failure, instead producing a ductile axial-flexural failure mode;

(4) Increase confinement and improve the lap splice strength; and,

(5) Increase the flexural strength of the column.

2.4 REVIEW OF MODELS OF CONFINED CONCRETE BEHAVIOR

As noted earlier, this research focuses on retrofit for axial-flexural and ductile shear failures. In both cases the design objective is to increase the confinement of the core concrete. This increased confinement will increase concrete strength and strain capacity, and will help to restrain the buckling of the longitudinal reinforcing bars. Increased concrete strain capacity, and a delay in buckling of the longitudinal bars will result in increased column ductility. Because the design objective in the retrofit is to increase confinement concrete confinement models are briefly reviewed here.

Several definitions need to be established at this point. Confinement may be either active or passive. Active confinement is defined as the case where the system is prestressed (i.e., confining material in tension and concrete in compression) prior to the application of significant axial stress. In passive confinement, confining pressures are engaged by transverse dilation of the concrete against the confining material. The transverse dilation is caused by the application of axial stress. Confinement may also be constant or variable. As these terms suggest, constant confinement means constant pressure is present as the axial stress is increased, and variable confinement means variable pressure is present as the axial stress is increased.
True constant confinement is obtained when confining pressures are applied to a specimen using a hydraulic pressure. In many instances, constant confinement is approximated by steel systems that use transverse steel reinforcement, because the steel provides constant confinement after yielding. Such a steel constant confinement system is also a passive system in that the lateral expansion of the concrete against the steel mobilizes the confinement. Variable confinement is obtained from confinement materials that do not yield. FRP jackets are a good example of this. In such systems, continued lateral expansion of the concrete core will generate continued increases in confining pressures in the concrete.

The following sections review existing confinement models.

2.4.1 Constant Confining Pressure Models

Richart et al. (1928) subjected concrete cylinders to a hydraulically applied constant confining stress and increasing axial stress. The following relationship between confining stress, $f_{con}$, and maximum principal compressive stress, $f_{cc}$, was determined:

$$f_{cc} = f_{c0} + 4.1 f_{con}$$

(2.1)

where: $f_{c0}$ = the unconfined concrete strength.

The strain corresponding to the peak stress, $f_{cc}$, is $\varepsilon_{cc}$, obtained by the following relationship:

$$\varepsilon_{cc} = \varepsilon_c (5(f_{cc}/f_{c0}) - 4)$$

(2.2)

where: $\varepsilon_c$ = the axial strain at the peak stress, $f_{c0}$, of the unconfined concrete.

Newman and Newman (1971) modified Richart's equation and proposed the following relationship:
A number of stress-strain models of concrete subjected to biaxial and triaxial stress have been proposed (e.g.: Kupfer et al. 1969, and Lui et al. 1972).

2.4.2 Models for the Confined Stress-Strain Behavior of Concrete Confined by Steel Reinforcement

For reinforced concrete columns, a number of tests of large scale column specimens have been carried out, and confined concrete response models have been proposed based on the results of these tests. Figure 2.2 shows the uniaxial stress-strain models proposed by Park, Kent and Sampson (1972), later modified by Park, Priestley and Wayne (1982), Sheikh and Uzumeri (1980), Vallenas, et al. (1977), and Mander et al. (1988). All of these models are based upon a constant confining pressure defined by the yielding of confining steel. Constant confining pressure models are generally considered appropriate when concrete is confined by steel, which provides a constant confining pressure following yielding of the steel.

2.4.3 Models for the Confined Stress-Strain Behavior of Concrete Confined by FRP Jackets

Restropol and DeVino (1996) use the model of confined concrete behavior proposed by Mander et al. (1988) to predict the behavior of reinforced concrete confined with FRP jackets. Restrepol and DeVino also proposal a definition of the “confined core concrete” for square (or rectangular) columns with external jackets. The confined core is equal to the gross section inside the confinement less the hemispherical regions of unconfined concrete along the sides of the column as shown in Figure 2.3. This definition is similar to the assumed region of confinement provided by conventional column ties. Additionally, the confining effects of both longitudinal and transverse internal reinforcement are included.
Saadatmanesh, Ehsani, and Li (1994) investigated the use of transverse FRP straps rather than a continuous jacket as a means to confine concrete. They presented a model of concrete behavior for this case which is similar to the previous model in that it uses the stress-strain relationship proposed by Mander et al. (1988), and assumes a constant level of confinement. Their approach is analogous to that used to model the confining behavior of conventional steel spirals.

2.4.4 Variable Confinement Models

Models of concrete stress-strain behavior when subjected to variable confining pressure are required when concrete is confined by a linear elastic material such as an FRP. The confinement pressure increases as the dilation of the column increases and the FRP jacket is engaged. Several investigations of the stress-strain behavior of concrete confined with FRP materials have been reported.

Fardis and Khalili (1982) proposed a simple hyperbolic stress-strain relationship having an initial slope equal to the Young’s modulus of unconfined concrete and passing through the ultimate stress and strain values proposed by Richart et al. (1928) discussed earlier (Equations 2.1 and 2.2). The ultimate stress and strain of the confined concrete are obtained by using the ultimate confining stress that the confinement material can provide in the calculations.

Hoppel et al. (1994) extended the concrete model proposed by Newman and Newman (1971). The model they propose applies Poisson’s ratio for concrete to determine the strain, and thus the stress in the confining jacket. Once the confining stress is determined the confined concrete strength is computed using Equation 2.3. The model tends to underestimate the experimental data presented, which may be caused by their assumption that the Poisson ratio of concrete is constant (Harries et al., 1997).

Labossiere et al. (1992) presented two models for the behavior of concrete confined with FRP jackets. The first model is a simple elastic-perfectly plastic representation of the concrete.
material. This simplification is justified in that it reasonably represents the observed behavior of circular FRP confined concrete elements (Deniauld, 1994). The second model is a non-linear model based on Saenz’s equation for the stress-strain behavior of concrete (Saenz, 1964) and considers the variable nature of Poisson’s ratio (Elwi and Murray, 1979). The acknowledgment that Poisson’s ratio varies with the level of principal stress is of upmost importance when determining the effect of passive confinement.

Rochette and Labossiere (1996) proposed an incremental finite element model for predicting the stress-strain response of concrete confined with carbon FRP (CFRP) materials. The model is based on an elastic-perfectly plastic concrete behavior model and a linear confining material model. Despite the apparent simplifications of the model, it is shown to agree well with experimental data in terms of stress versus both axial and transverse strain. It is noted that the experimental data presented are based on 152 mm square column stubs having very large amounts of CFRP confinement. The elastic-perfectly plastic assumption of concrete behavior may not be as appropriate for smaller amounts of volumetric confinement (Harries et al., 1997).

Ahmad and Shah (1982) and Madas and Elnashai (1992) proposed an approach for determining the concrete stress-strain relationship for the case where transverse steel reinforcement has not yet yielded, that is, for conditions of variable confining pressure. The approach determines instantaneous confinement conditions, treating these as constant confinement. By integrating through successively greater amounts of constant confinement, a relationship for concrete experiencing increasing variable confinement is developed. In order to determine the individual constant confinement states, the relationship between axial and transverse strains proposed by Elwi and Murray (1979) is used to determine transverse strains which engage confining pressures. A schematic representation of developing the variably confined concrete stress-strain relationship is shown in Figure 2.4.
Mirmiran and Shahawy (1995) proposed a model for the confined behavior of concrete which adapts the model proposed by Madas and Elnashai (1992) to use with a linear elastic confining material. The relationship between principle and transverse strains used was again the one proposed by Elwi and Murray (1979). As yet, no experimental verification of the model is available.

Demers et al. (1994), Rochette and Labboissiere (1996), and Deniauld (1994) all report a noticeable difference in the stress-strain response of circular vs. square elements. The circular elements exhibited an almost bi-linear stress-strain curve, whereas the square elements have an ascending branch, an abrupt decrease in load carrying capacity, and finally a slow increase in load carrying capacity as more confinement is engaged due to continued dilation of the concrete. This variance in stress-strain curves is a result of what is defined as a shape factor effect and can be reduced by rounding the corners of square members. This effect was demonstrated by Rochette (1996). Figure 2.5 illustrates the differences in the stress-strain curves created by shape factor effects.

Harmon and Slattery (1992) present data that indicates that cycling axial loads has little effect on the backbone monotonic response. Howie and Karbhari (1994) report that fibers oriented in the hoop direction are most important to providing good confinement and that crossing fibers may weaken the jacket and make it susceptible to a brittle progressive failure.

2.4.5 Variably Confined Concrete Model

The Variably Confined Concrete Model (VCCM) developed by Harries et al. (1997) is used in this investigation. The VCCM model is a modified version of the method proposed by Madas and Elnashai (1992) used in conjunction with the model of confined concrete behavior proposed by Mander et al. (1988) to develop the confined concrete stress-strain relationship.
Stress-strain curves developed using the VCCM model are well representative of axial test data presented in Kestner et al. (1997).

The VCCM uses an iterative procedure for predicting the stress-strain relationship for confined concrete. A schematic of this procedure is shown in Figure 2.4. In order to use this procedure the following four relationships are established (see Figure 2.4).

quadrant 1: stress-strain relationship for concrete confined with a constant confining pressure;

quadrant 2: relationship between axial and transverse strains, related to Poisson’s ratio for concrete;

quadrant 3: stress-strain relationship for confining material; and,

quadrant 4: relationship between stress in confining material and confining pressure provided.

The procedure begins by selecting an axial strain and proceeding through each of the established relationships to determine a confining pressure corresponding to that strain. The confined stress-strain relationship for that confining pressure is calculated and the predicted stress corresponding to the initially selected axial strain is determined. The entire passive confinement stress-strain relationship is found by incrementing axial strains and plotting a backbone curve through the individually determined stress-strain curves for constant confinement as shown in Figure 2.4.

2.5 RETROFIT OF NON-DUCTILE REINFORCED CONCRETE COLUMNS USING STEEL OR REINFORCED CONCRETE JACKETS

2.5.1 Experimental Behavior of Steel Jacketed Reinforced Concrete Columns

Steel jacket retrofits are recognized as a viable method of retrofitting reinforced concrete columns by increasing their strength and ductility. Two investigations, in particular, are noteworthy of this retrofit practice.
Aboutaha, Engelhardt, Jirsa and Kreger (Aboutaha, 1994 and Aboutaha et al. 1994 and 1996) present data from a number of tests involving rectangular columns retrofit with steel jackets conducted at the University of Texas at Austin. Non-ductile reinforced concrete column failure modes, including lap splice failures and shear failures were investigated. The tests were conducted in the absence of axial load, and although this may represent a critical condition, there is some evidence to suggest lap splices may not fail if the axial load is included (Lynn et al. 1996). In addition, the shear critical column specimens had a shear span ratio, a/d, of only 1.33, where “a” is the shear span - a region of constant shear from zero moment to the point of maximum moment and “d” is the depth of the cross section. It is likely that the entire specimen may be considered a “disturbed region” under these conditions (Harries et al., 1997). The non-retrofit columns exhibited non-ductile failures. Retrofit shear and lap splice columns exhibited higher strength, ductility, and energy dissipation, resulting in an improved overall response. Therefore, retrofit of non-ductile concrete columns using steel jackets can enhance the response of columns with poor shear and lap splice details.

Chai, Priestley, and Seible (1991 and 1994) conducted an investigation of bridge piers retrofit with steel jackets. Steel jacket retrofits of a hinge region of concrete columns were conducted and the results of the study indicate a moderate strength increase and a notable increase in the ductility capacity. In general, the steel jackets also affected a change in the mode of failure from less desirable brittle modes to more ductile modes. The use of oval steel jackets on the rectangular columns enabled a higher confining pressure. As such, the oval steel jacket retrofits perform significantly better than steel plate jackets such as those tested by Aboutaha (1994). It has been proposed that the only way steel jackets may be effectively used is in an ovoid form, which may make them inappropriate for building column. Results of this program indicate an increase
in lateral stiffness of the column, as well as, an increase in the ductility. Two limit states were identified:

(1) A limit state corresponding to the ultimate compressive strain of confined concrete; and,

(2) A limit state corresponding to low-cycle fatigue of the longitudinal steel.

Due to low-cycle fatigue of the longitudinal steel under reverse cyclic loading, it is recommended that the extreme longitudinal tension strain should be limited to 75% of the ultimate tensile strain of the longitudinal steel.

Table 2.3 summarizes the above investigations of the response of steel jacket retrofits of reinforced concrete columns.

The data presented in Table 2.3, and 2.7 through 2.9 are defined as follows:

- $d \times b$ column dimensions, where the lateral load is applied parallel to the $d$ dimension.
- $\rho_s$ longitudinal reinforcement ratio;
- $\rho_t$ transverse steel reinforcement ratio = $A_t/sb$ for rectangular; and = $4A_t/D's$ for circular columns;
- $a/d$ shear span ratio = $M/Nd$;
- $\text{history}$ loading history applied (RC-3 = reversed cyclic with 3 cycles per load/displacement level)
- $\text{capacity}$ ratio of peak lateral load observed to that observed for the corresponding as-built specimen
- $\mu_{\text{max}}$ maximum observed displacement ductility = $\text{DR}_{\text{max}} / \text{DR}_{\text{yield}}$; and
- $\mu_{80}$ displacement ductility corresponding to 20% reduction in lateral load carrying capacity.

### 2.5.2 Experimental Behavior of Concrete Jacketed Reinforced Concrete Columns

Bett et al. (1988) report the results of lateral load response tests of strengthened and repaired reinforced concrete columns. Three columns were constructed at two-thirds scale with
reinforcement details typical of the 1950's and 1960's. The columns were 305 mm (12 in.) square by 915 mm (36 in.) in height. One column was tested without retrofit and performed in a non-ductile manner. The retrofit technique included the addition of closely spaced transverse ties, secured to additional longitudinal reinforcing bars. After the reinforcing cages were in place, the column was shotcreted with an additional 64 mm (2.5 in.) of concrete. The additional longitudinal reinforcing bars were not cast into the footing; therefore, they did not provide additional flexural strength to the column. The retrofit specimens, as well as the repaired column performed better than the original column. Columns strengthened by the concrete jacket showed an increase in strength and stiffness. The repaired column performed almost as well as the retrofit specimens. Overall the ductility of the columns was enhanced through the use of concrete jackets with additional transverse ties.

Mitchell et al. (1988) report case studies on the use of reinforced concrete jackets to repair and rehabilitate reinforced concrete building columns damaged in the 1985 Mexico City earthquake. It was reported that concrete jackets were highly effective when both the column and beam suffered significant damage. The concrete jacket could be placed around both the beams and columns resulting also in an improved connection.

2.6 RETROFIT OF NON-DUCTILE REINFORCED CONCRETE COLUMNS USING FRP JACKETS

2.6.1 FRP Materials

Fiber reinforced polymer (FRP) materials are composites consisting of high performance fibers (typically, carbon, glass or aramid) in a polymer matrix. For typical structural applications, fibers may take the form of a continuous unidirectional mat (called a tow sheet), stitched or woven fabrics having single or multiple fiber orientation, or mats of chopped fiber having random orientation. The polymer matrix may be an epoxy, vinylester, or polyester resin. Typically, only
the material properties and orientations of the fibers are considered in determining the properties of the composite FRP.

2.6.1.1 Advantages and Disadvantages of the use of FRP Materials

FRP jackets have demonstrated potential for retrofitting non-ductile concrete columns (Kestner et al., 1997). FRP jackets have several advantages over steel and concrete jackets. FRP materials have large strength-to-weight and stiffness-to-weight ratios and may have their directional properties easily tailored to suit a particular application. Their high strength can be used to generate large confining pressures in concrete columns. Due to their strength-to-weight ratio and potential anisotropy, FRP jacket retrofits do significantly increase the member weight, stiffness, or dimensions which could potentially increase the seismic forces on the columns. Additionally, the low weight of the material makes them easy to apply, which in turn decreases labor costs and reduces disturbance of the use and occupancy of the structure during installation.

Although FRP jackets have several advantages, the material also has a few disadvantages. FRP materials can be susceptible to ultraviolet radiation, chemical exposure, moisture permeability, and extreme ambient temperature. FRP materials possess thermal properties that result in decreased tensile strength with increasing temperatures. Fire resistance and toxicity of certain resin formulations are also concerns which need to be addressed. Another disadvantage of FRP materials is that they have shown themselves susceptible to stress fatigue failures. Additionally, some FRP materials are currently relatively expensive. However, this cost may be offset by decreases in labor and life-cycle costs.

Clearly, FRP materials have many advantages. Therefore, it is important to further investigate the properties of these materials in order to understand their differences in relation to conventional steel and concrete materials.
2.6.1.2 Stress-Strain Properties of FRP Materials

Physical properties of FRP materials are dependent upon the properties of their components, the orientation and distribution of these components, the method of fabrication, and the environment to which they are exposed.

Although the values of ultimate stress and modulus of elasticity vary with the type of material and number of plies, the general tensile stress-strain response of FRP materials in their principal orientation can be modeled as a linear-elastic behavior.

2.6.1.3 Failure of FRP Materials

Theoretical models defined by the linear-elastic stress-strain diagram generally assume that all fibers fail at the same strain. However, the failure strain of the fibers is not a unique quantity and is instead characterized statistically (ASM International Handbook, 1987). Thus individual fibers fail at various strain levels. As the number of failed fibers within a cross-section increases, some region in the cross-section becomes too weak to support increased load, and the laminate ruptures along this zone of weakness. Strength and stiffness values of the FRP material thus differ from that of the fiber reinforcement. Such disparities are influenced by (Argarwal and Boutman, 1990):

(1) Misaligned fibers;
(2) Fibers of nonuniform strength;
(3) Discontinuous fibers;
(4) Interfacial conditions; and,
(5) Residual stresses.

FRP materials generally experience internal failures well before the material itself fractures. Internal failures consist of fiber rupture, matrix microcracking, debonding noted by fiber separation from the matrix, and delamination defined as separation of laminae from one another.
One or more of these internal failure mechanisms may contribute to the overall FRP failure (Argarwal and Broutman, 1990).

2.6.2 Behavior of Non-Ductile Reinforced Concrete Columns with FRP Jackets

Several axial and lateral load tests have been performed on columns retrofit with FRP jackets. The results of these tests are summarized below and presented in Tables 2.4 through 2.8.

2.6.2.1 Axial Behavior

In addition to the material presented in this section, Harries et al. (1997) present a thorough overview of the use of FRP materials for rehabilitation and retrofit of reinforced concrete columns. Several investigations have been conducted on the axial response of concrete confined with FRP materials. The results of these investigations are summarized in Tables 2.4 through 2.6. It is noted that Demers (1994) and Kestner et al. (1997) are the only two investigations that consider the axial response of reinforced concrete columns. All of the other tests involve plain concrete columns.

Demers (1994) tested sixteen 300 mm (12 in.) diameter, 1.2 m (4 ft.) tall reinforced concrete columns. The goal of this investigation was the retrofit of existing columns. For this reason, the amount of longitudinal and transverse reinforcing steel was varied to simulate corrosion effects. Half of the 16 columns tested were loaded to an axial stress approximately equivalent to $f'_c$ before they were jacketed with the carbon sheets. All of the jacket retrofits were designed to provide a minimum confining pressure of 5 MPa (725 psi), resulting in the use of three plies of pre-impregnated carbon fiber sheets. The previously undamaged columns achieved maximum strength and deformation capacity enhancements of 1.17 to 3.00 times those of the unjacketed specimens. A repaired column which had been significantly damaged was able to exceed its original load capacity. Demers noted that the carbon FRP jacket ruptured at significantly lower strains than the manufacturer’s indicated material rupture strain of 0.015 mm/mm. Results of Demers’ individual tests are given in Table 2.4.
Kestner et al. (1997) tested a series of 152 mm round and square, by 610 mm tall (6 x 24 in.) plain concrete specimens, having external FRP jackets and a series of eight 1.83 m (6 ft) tall reinforced concrete columns having 508 mm (20 in.) diameter circular and 450 mm (18 in.) square cross-sections. Jacket retrofit materials were varied as well as the number of plies of the material. The dilation of the column was shown to decrease with an increase in confining pressure, influenced by FRP jacket stress and stiffness. Cross section geometry was also shown to significantly influence a member’s stress-strain response. It was found that improved axial stress-strain responses were obtained for all jacketed specimens. Results of the full scale reinforced concrete columns tested by Kestner et al. are given in Table 2.5. Results from other axial load tests are presented in Table 2.6.

2.6.2.2 Lateral Behavior

Several recent studies have been conducted on reinforced concrete columns using FRP jacket retrofits. In general, the FRP fibers are oriented in the transverse direction (0°), although some retrofit jackets have included additional material oriented in the longitudinal (90°) direction. The results of significant lateral test studies are summarized in Tables 2.7 and 2.8.

Ballinger, Maeda, and Hoshijima (1992) report a number of Japanese tests of lap spliced bridge piers retrofit with carbon FRP (CFRP) jackets having various transverse and longitudinal orientations. The reinforced concrete specimens have exceptionally long lap splices which are not critical. Indeed, one retrofit investigation (Specimen 2) provided a bi-directional jacket above the lap splice region but not covering the region itself. This technique resulted in reducing the flexural response of the region above the splice and causing a failure at the top rather than the bottom of the splice. The other retrofit jackets confined the entire column and provided moderate strength and significant ductility enhancement. Specimen 5, despite lacking axial load, did not exhibit as significant an increase in capacity as Specimen 4, which had essentially the same confinement but
had a small applied axial load. This discrepancy is likely a result of the large splice region of Specimen 5 resulting in twice the longitudinal steel in the lower regions of the column, creating a less ductile response.

Jin, Saadatmanesh, and Ehsani (Jin, 1995, Jin et al. 1994, and Saadatmanesh et al. 1992, 1994, 1996) conducted several tests which included the effect of several variables. The specimens tested consisted of circular and rectangular columns and the effect of active versus passive confinement was also investigated. Active pressure was provided by slightly over-sizing the jackets and injecting pressurized epoxy in the gap between the concrete and jacket, effectively prestressing the jacket. The prestressed jackets were expected to enhance the performance of the column at lower ductility levels, before significant transverse strains are developed. Prestressed jackets did not appear to significantly effect the overall response of the retrofit columns. Due to the lack of travel in the test set up, few of the retrofit columns were taken to failure, although significant ductility capacity increases were obtained. Another important characteristic of this study was the retrofit of the previously tested “as-built” specimens. All of the retrofit columns exhibited an increase in ductility capacity and shear resistance. These results are evidenced by the ductile failure modes of the retrofit columns.

Osada, Ono, Yamaguchi, and Ikeda (1996) report a number of bi-directional CFRP jacket retrofit tests. Initial monotonic tests were conducted to determine basic response characteristics of the jacketed columns. Dynamic tests were carried out using ground motion records from the 1995 Kobe earthquake. Two dynamic tests were conducted on each specimen. The first test, having an equivalent peak ground acceleration (pga) of 0.2g, investigated the near yield response of the structure. The second test, having an equivalent pga of 0.4g or 0.45g, investigated the ultimate response of the structure. Results of the study indicate an increase in the ductility.
Stanton, McRae, and Nosho (1996) conducted tests at the University of Washington that represent the only existing study specifically representing building, rather than bridge, columns. The building considered was a parking garage. Three CFRP retrofits were demonstrated, representing "heavy", "moderate", and "light" retrofits (Specimens 2, 3 and 4, respectively). Each retrofit demonstrated a similar capacity increase. Only the "heavy" retrofit exhibited a moderate increase in ductility capacity. It is noted, however, that the combined moderate strength and ductility increases may prove sufficient to protect the column against moderate seismic attack. All of the column failures were associated with rupture of the FRP jacket. Additionally, Seible and Innomorato's (1995) design method was modified for the application of FRP materials in seismic retrofit using the rupture of the FRP jacket as the design limit state. The Seible and Innomorato design procedure is discussed further in Chapter 3 of this report.

A significant number of CFRP and glass FRP (GFRP) jacket retrofit column tests have been conducted by Priestley, Seible, and Fyfe (1992) at the University of California at San Diego (UCSD). This program has been the primary proof tests of jacket retrofits for acceptance by the California Department of Transportation (CALTRANS). The program also generated the design and detailing guidelines for bridge pier retrofits used by CALTRANS (Seible and Innamorato, 1995) and summarized in Chapter 3 of this report. As a result of these studies, a number of bridge piers have been retrofit in California as demonstration projects using steel, CFRP and GFRP jackets. The results of this investigation are summarized in Table 2.8.

Carbon fiber retrofits were all applied using XXsys Technologies' RoboWrapper (XXsys, 1996) which incorporates the use of a resin impregnated (so called, prepreg) carbon. Using the RoboWrapper allowed jackets of variable thickness to be easily and accurately applied. The CFRP jackets performed very well, exhibiting moderate strength and significant ductility increases as well as affecting the final mode of failure exhibited. The responses of the steel and CFRP jackets
were very similar, exhibiting excellent energy dissipation. Notably, the rectangular columns retrofit with rectangular CFRP jackets performed almost as well as the rectangular columns retrofit with oval steel jackets, suggesting the FRP jackets may be more appropriate for building structures. Despite the improved response of rectangular jacket retrofits, rupture of the jacket occurred at its corners. This mode of failure is quite brittle although it does occur only at large transverse strains and only after significant amounts of energy have been dissipated. Oval CFRP jacket retrofits were also shown to behave quite well. Individual test results for the CFRP retrofit columns are given in Table 2.8.

Glass fiber retrofits were applied using hand lay-up techniques using a woven E-Glass fabric. The fabric provides fibers in only one direction. Various thicknesses of GFRP material were applied in discrete layers. The GFRP retrofit columns exhibited moderate increases in load carrying ability and significant increases in ductility capacity, and affected the mode of failure. Again, rupture of the jacket, followed by buckling of the longitudinal reinforcing appears to be the typical behavior exhibited by rectangular retrofit columns. This failure, however, does not occur before significant ductility and energy dissipation has been exhibited. Individual test results for the CFRP retrofit columns are given in Table 2.9.

Of note are a few early GFRP column retrofit demonstration projects installed in California. Each of these early installations exhibited no damage due to the 1994 Northridge Earthquake (Hexcel-Fyfe Company, 1996):

(1) Southbound Interstate 5 to Eastbound Highway 2 Ramp: twelve 1830 mm dia. columns;
(2) Northbound Interstate 5 Griffith Park Exit Ramp: three 1220 mm dia. columns;
(3) Northbound Interstate 5 to Westbound Highway 2 Ramp: two multi-column bents;
(4) Northbound Highway 101 (Santa Barbara): two 1525 mm dia. columns; and
(5) Nikko Hotel, Beverly Hills: 34 rectangular columns.
The Interstate 5 installations are 32 km from the Northridge epicenter, the Nikko Hotel is 22 km from the epicenter. The Highway 101 retrofit is 336 km from Northridge and experienced little ground motion. A significant number of steel jacketed columns also successfully withstood the 1994 Northridge earthquake.
Table 2.1 Global and local ductility requirements corresponding to different drift limits for structures analyzed by Wu (1995).

<table>
<thead>
<tr>
<th>Maximum Mechanism Drift</th>
<th>Global Displacement Ductility</th>
<th>Column Curvature Ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>1%</td>
<td>2.3</td>
<td>4.3</td>
</tr>
<tr>
<td>2%</td>
<td>4.4</td>
<td>9.9</td>
</tr>
</tbody>
</table>
### Table 2.2 Summary of ACI 318 Code provisions for non-ductile reinforced concrete columns.

<table>
<thead>
<tr>
<th>Equations given in units of MPa and mm</th>
<th>ACI Building Code Requirements for Reinforced Concrete (ACI 318-year)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>General Requirements for Columns</strong></td>
<td></td>
</tr>
<tr>
<td>minimum tie size</td>
<td>#2</td>
</tr>
<tr>
<td>maximum tie spacing</td>
<td>s ≤ 16d_b, 48d_d and least dimension of column, d</td>
</tr>
<tr>
<td>design for shear</td>
<td>V_u = V_s + V_i; V_i = A_i f_d/s; shear reinforcement required if V_u &gt; 0.5V_c</td>
</tr>
<tr>
<td>minimum shear reinforcement</td>
<td>A_s ≥ b w s/3f_y if shear reinforcement is required</td>
</tr>
<tr>
<td>maximum spacing for shear design</td>
<td>d/2 ≤ 600 mm for V_s &lt; 0.33 ( \sqrt{f_y} ) b_w d</td>
</tr>
<tr>
<td></td>
<td>d/4 ≤ 300 mm for V_s ≥ 0.33 ( \sqrt{f_y} ) b_w d</td>
</tr>
<tr>
<td>lateral support of longitudinal bars</td>
<td>all longitudinal bars confined by tie having 90° bend</td>
</tr>
<tr>
<td></td>
<td>alternate and corner longitudinal bars confined with ties having enclosed angles less than 135°, provided spacing between longitudinal bars is less than 150 mm.</td>
</tr>
<tr>
<td>minimum splice length</td>
<td>none</td>
</tr>
</tbody>
</table>

#### Provisions for Seismic Design

| length of plastic hinge region        | not less than d, 450 mm or 1/6 clear height |
| maximum spacing in hinge region       | s ≤ 100 mm; s ≤ d/4 and 100 mm               |
| maximum spacing elsewhere             | s ≤ d/2; general prov. apply                 |
|                                       | s ≤ 6d_b and 150 mm                         |
| minimum shear reinforcement           | A_s ≥ 0.22(l_s f_y'/f_y)[A_s/A_s+1]         |
|                                       | A_s ≥ 0.3(sh_f_y'/f_y)[A_s/A_s+1]; A_s ≥ 0.09(sh_f_y'/f_y) |
| anchorage of ties                     | seismic hoops, having 135° anchorages required over length of plastic hinge |
| minimum splice length                 | 400 mm; 300 mm                             |
Table 2.3 Experimental investigations of reinforced concrete columns with steel jacket retrofits.

<table>
<thead>
<tr>
<th>Researcher ID</th>
<th>specimen details</th>
<th>loading</th>
<th>retrofit details</th>
<th>capacity</th>
<th>ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>d x b (mm)</td>
<td>$\rho_s$</td>
<td>$\rho_c$</td>
<td>splice</td>
<td>a/d ratio</td>
</tr>
<tr>
<td>Aboutaha 1994</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F1</td>
<td>450 x 900</td>
<td>0.0008</td>
<td></td>
<td></td>
<td>24$b$</td>
</tr>
<tr>
<td>F2</td>
<td>450 x 900</td>
<td>0.0008</td>
<td></td>
<td></td>
<td>24$b$</td>
</tr>
<tr>
<td>F4</td>
<td>450 x 900</td>
<td>0.0005</td>
<td></td>
<td></td>
<td>24$b$</td>
</tr>
<tr>
<td>F5</td>
<td>450 x 900</td>
<td>0.0008</td>
<td></td>
<td></td>
<td>24$b$</td>
</tr>
<tr>
<td>F6</td>
<td>450 x 900</td>
<td>0.0005</td>
<td></td>
<td></td>
<td>24$b$</td>
</tr>
<tr>
<td>F8</td>
<td>450 x 900</td>
<td>0.020</td>
<td></td>
<td></td>
<td>24$b$</td>
</tr>
<tr>
<td>F9</td>
<td>450 x 900</td>
<td>0.0005</td>
<td></td>
<td></td>
<td>24$b$</td>
</tr>
<tr>
<td>F11</td>
<td>450 x 900</td>
<td>0.0008</td>
<td></td>
<td></td>
<td>24$b$</td>
</tr>
<tr>
<td>F12</td>
<td>450 x 900</td>
<td>0.0008</td>
<td></td>
<td></td>
<td>24$b$</td>
</tr>
<tr>
<td>F14</td>
<td>450 x 675</td>
<td>0.0008</td>
<td></td>
<td></td>
<td>24$b$</td>
</tr>
<tr>
<td>F15</td>
<td>450 x 675</td>
<td>0.0010</td>
<td></td>
<td></td>
<td>24$b$</td>
</tr>
<tr>
<td>F16</td>
<td>450 x 675</td>
<td>0.0008</td>
<td></td>
<td></td>
<td>24$b$</td>
</tr>
<tr>
<td>F17</td>
<td>450 x 675</td>
<td>0.0010</td>
<td></td>
<td></td>
<td>24$b$</td>
</tr>
</tbody>
</table>
Table 2.3 (continued) Experimental investigations of reinforced concrete columns with steel jacket retrofits.

<table>
<thead>
<tr>
<th>Researcher</th>
<th>ID</th>
<th>Specimen details</th>
<th>loading</th>
<th>retrofit details</th>
<th>capacity</th>
<th>ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aboutaha 1994 (continued)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td>450 x 900</td>
<td>0.0008</td>
<td>none</td>
<td>1.33</td>
<td>none - as built</td>
<td>1.00</td>
</tr>
<tr>
<td>S2</td>
<td></td>
<td>0.0005</td>
<td>none</td>
<td>1.33</td>
<td>bolted collars</td>
<td>1.05</td>
</tr>
<tr>
<td>S3</td>
<td>900 x</td>
<td>0.0008</td>
<td>none</td>
<td>RC-2</td>
<td>none - as built</td>
<td>1.00</td>
</tr>
<tr>
<td>S4</td>
<td></td>
<td>0.0005</td>
<td>none</td>
<td>1.33</td>
<td>none - as built</td>
<td>1.00</td>
</tr>
<tr>
<td>S5</td>
<td></td>
<td>0.020</td>
<td>none</td>
<td>RC-2</td>
<td>bolted collars</td>
<td>1.33</td>
</tr>
<tr>
<td>S6</td>
<td>450 x 900</td>
<td>0.0005</td>
<td>none</td>
<td>RC-2</td>
<td>welded 6.35 mm jacket</td>
<td>1.69</td>
</tr>
<tr>
<td>S7</td>
<td>900 x</td>
<td>0.0005</td>
<td>none</td>
<td>RC-2</td>
<td>shop welded - field bolted jacket</td>
<td>1.63</td>
</tr>
<tr>
<td>S8</td>
<td></td>
<td>0.010</td>
<td>none</td>
<td>RC-2</td>
<td>U shaped welded jacket with through bolts</td>
<td>1.63</td>
</tr>
<tr>
<td>S9</td>
<td></td>
<td>0.0010</td>
<td>none</td>
<td>RC-2</td>
<td>none - as built</td>
<td>1.00</td>
</tr>
<tr>
<td>SI0</td>
<td>900 x 450</td>
<td>0.0010</td>
<td>none</td>
<td>RC-2</td>
<td>welded jacket</td>
<td>2.14</td>
</tr>
<tr>
<td>S11</td>
<td>450 x</td>
<td>0.0010</td>
<td>none</td>
<td>RC-2</td>
<td>C shaped welded jacket with anchor bolts</td>
<td>1.29</td>
</tr>
</tbody>
</table>
Table 2.3 (continued) Experimental investigations of reinforced concrete columns with steel jacket retrofits.

<table>
<thead>
<tr>
<th>Researcher</th>
<th>ID</th>
<th>Specimen details</th>
<th>loading</th>
<th>retrofit details</th>
<th>capacity</th>
<th>ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chai et al. 1991</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td>1.00</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td>1.23</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>610 dia.</td>
<td>0.025</td>
<td>none</td>
<td>1.00</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td></td>
<td>0.0017</td>
<td>none</td>
<td>1.15</td>
<td>8.0</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td>1.44</td>
<td>8.0</td>
</tr>
<tr>
<td>1-R</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.17</td>
<td>6.0</td>
</tr>
</tbody>
</table>
Table 2.4 Experimental results of axial columns confined with CFRP jackets (Demers 1994).

<table>
<thead>
<tr>
<th>ID</th>
<th>( f' ) (MPa)</th>
<th>( \rho_n )</th>
<th>( \rho_v )</th>
<th>preloaded</th>
<th>( f_{cc} ) (MPa)</th>
<th>( e_{cc} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.0071</td>
<td>0.0017</td>
<td>no</td>
<td>32.2</td>
<td>0.0038</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>0.0141</td>
<td>0.0053</td>
<td>yes</td>
<td>31.3</td>
<td>0.0077</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>0.0034</td>
<td>yes</td>
<td>36.6</td>
<td>0.0099</td>
<td></td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>0.0107</td>
<td>no</td>
<td>37.1</td>
<td>0.0070</td>
<td></td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>0.0355</td>
<td>no</td>
<td>37.0</td>
<td>0.0098</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>0.0107</td>
<td>yes</td>
<td>38.8</td>
<td>0.0091</td>
<td></td>
<td></td>
</tr>
<tr>
<td>G</td>
<td>0.0213</td>
<td>yes</td>
<td>51.1</td>
<td>0.0054</td>
<td></td>
<td></td>
</tr>
<tr>
<td>J</td>
<td>0.0034</td>
<td>no</td>
<td>50.1</td>
<td>0.0055</td>
<td></td>
<td></td>
</tr>
<tr>
<td>K</td>
<td>0.0141</td>
<td>no</td>
<td>52.3</td>
<td>0.0038</td>
<td></td>
<td></td>
</tr>
<tr>
<td>K</td>
<td>0.0107</td>
<td>yes</td>
<td>55.7</td>
<td>0.0059</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M</td>
<td>0.0213</td>
<td>no</td>
<td>54.8</td>
<td>0.0042</td>
<td></td>
<td></td>
</tr>
<tr>
<td>N</td>
<td>0.0034</td>
<td>yes</td>
<td>52.1</td>
<td>0.0049</td>
<td></td>
<td></td>
</tr>
<tr>
<td>O</td>
<td>0.0355</td>
<td>yes</td>
<td>52.8</td>
<td>0.0050</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P</td>
<td>0.0107</td>
<td>no</td>
<td>53.6</td>
<td>0.0056</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 2.5 Experimental results of full scale axial tests of reinforced concrete columns confined with CFRP and GFRP jackets (Kestner et al. 1997).

<table>
<thead>
<tr>
<th>ID</th>
<th>Retrofit Details</th>
<th>$f'_c$ (MPa)</th>
<th>$\rho_s$</th>
<th>$\rho_r$</th>
<th>$f_{cc}$ (MPa)</th>
<th>$\epsilon_{cc}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>none</td>
<td>24.6 (3570)</td>
<td></td>
<td>0.015</td>
<td>32.8 (4760)</td>
<td>0.0020</td>
</tr>
<tr>
<td>C2</td>
<td>multidirectional E-glass</td>
<td>23.9 (3470)</td>
<td></td>
<td>0.002</td>
<td>36.9 (5350)</td>
<td>0.0031</td>
</tr>
<tr>
<td>C3</td>
<td>unidirectional E-glass</td>
<td>26.3 (3820)</td>
<td></td>
<td>0.015</td>
<td>38.9 (5640)</td>
<td>0.0078</td>
</tr>
<tr>
<td>C4</td>
<td>unidirectional carbon</td>
<td>25.0 (3620)</td>
<td></td>
<td></td>
<td>50.0 (7250)</td>
<td>0.0113</td>
</tr>
<tr>
<td>S1</td>
<td>none</td>
<td>26.0 (3770)</td>
<td></td>
<td>0.001</td>
<td>31.5 (4570)</td>
<td>0.0019</td>
</tr>
<tr>
<td>S2</td>
<td>multidirectional E-glass</td>
<td>27.4 (3980)</td>
<td></td>
<td></td>
<td>36.4 (5290)</td>
<td>0.0016</td>
</tr>
<tr>
<td>S3</td>
<td>unidirectional E-glass</td>
<td>31.9 (4630)</td>
<td></td>
<td></td>
<td>35.5 (5150)</td>
<td>0.0025</td>
</tr>
<tr>
<td>S4</td>
<td>unidirectional carbon</td>
<td>31.1 (4510)</td>
<td></td>
<td></td>
<td>37.4 (5430)</td>
<td>0.0021</td>
</tr>
</tbody>
</table>
Table 2.6 Experimental studies of plain concrete columns under axial load with FRP jackets.

<table>
<thead>
<tr>
<th>Researcher</th>
<th>Size (mm)</th>
<th>Confine ment</th>
<th>$f'_c$ (MPa)</th>
<th>$f_c/f'_c$</th>
<th>$\epsilon_{ce}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Demers et al. 1996</td>
<td>150 dia. x 300</td>
<td>3-12 layers aramid tape; $E = 4.3$ kN/mm / layer $f_{ult} = 104$ N/mm / layer</td>
<td>44</td>
<td>1.0 - 1.65</td>
<td>0.018</td>
</tr>
<tr>
<td></td>
<td>150 x 150 x 500</td>
<td></td>
<td>1.14 - 1.25</td>
<td>0.003</td>
<td></td>
</tr>
<tr>
<td>Harmon and Slattery 1992</td>
<td>51 dia. x 102</td>
<td>1,2,3 and 7 layers CFRP; ($\rho = 0.007 - 0.054$) $E = 235$ GPa (fiber only); $f_{ult} = 3500$ MPa</td>
<td>41</td>
<td>2.09 - 5.87</td>
<td>0.01 - 0.035</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>103</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Howie and Karbhari 1994</td>
<td>150 dia. x 300</td>
<td>1 layer CFRP; $E = 77$ GPa; $f_{ult} = 1100$ MPa</td>
<td>38.6</td>
<td>1.16</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>2 layers CFRP</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>3 layers CFRP</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4 layers CFRP</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>CFRP having various orientations</td>
<td></td>
<td></td>
<td>1.02 - 1.77</td>
</tr>
<tr>
<td>Karbhari and Eckel 1994 and 1995</td>
<td>150 dia. x 300</td>
<td>2 layers GFRP</td>
<td>51.9</td>
<td>1.22 - 1.28</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2 layers CFRP</td>
<td></td>
<td>1.26 - 1.32</td>
<td>0.008</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2 layers Aramid</td>
<td></td>
<td>1.01 - 1.06</td>
<td>0.005</td>
</tr>
<tr>
<td>Karbhari and Eckel 1993</td>
<td>150 dia. x 300</td>
<td>2 layers GFRP</td>
<td>38.2</td>
<td>1.47</td>
<td>0.005</td>
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<td></td>
<td></td>
<td>4 layers GFRP</td>
<td></td>
<td>1.94</td>
<td>0.005</td>
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<tr>
<td></td>
<td></td>
<td>1 layer CFRP</td>
<td></td>
<td>1.85</td>
<td>0.006</td>
</tr>
<tr>
<td>Labossiere et al. 1992</td>
<td>150 dia. x 300</td>
<td>1 layer GFRP</td>
<td>32</td>
<td>1.00</td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3 layers GFRP</td>
<td></td>
<td>1.50</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1 layer CFRP</td>
<td></td>
<td>1.25</td>
<td>0.015</td>
</tr>
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</table>
Table 2.6 (continued) Experimental studies of plain concrete columns under axial load with FRP jackets.

<table>
<thead>
<tr>
<th>Researcher</th>
<th>Size (mm)</th>
<th>confinement</th>
<th>( f'_c ) (MPa)</th>
<th>( f_{ce}/f'_c )</th>
<th>( e_{ce} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nanni et al. 1992</td>
<td>150 dia. x 300</td>
<td>braided Aramid tape having varying strengths and applied at various pitches</td>
<td>40</td>
<td>1.13 - &gt;1.75</td>
<td>0.005 - &gt;0.013</td>
</tr>
</tbody>
</table>
Table 2.7 Experimental investigations of reinforced concrete columns with FRP jacket retrofits.

<table>
<thead>
<tr>
<th>Researcher</th>
<th>ID</th>
<th>Specimen details</th>
<th>loading</th>
<th>retrofit details</th>
<th>capacity</th>
<th>ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>d x b mm</td>
<td>ρs</td>
<td>ρv</td>
<td>splice</td>
<td>a/d ratio</td>
</tr>
<tr>
<td>Ballinger et al., 1992</td>
<td>1</td>
<td>0.0112 0.023 in</td>
<td>57d_b</td>
<td>0.03A&lt;f_b&gt; 0.03A&lt;f_c&gt;</td>
<td>RC-3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.025 0.0005</td>
<td>none</td>
<td>7.0</td>
<td>0.03A&lt;f_b&gt; 0.03A&lt;f_c&gt;</td>
<td>RC-3</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>305 dia.</td>
<td>0.025</td>
<td>none</td>
<td>0.03A&lt;f_b&gt; 0.03A&lt;f_c&gt;</td>
<td>RC-3</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>368 x 241</td>
<td>0.027</td>
<td>none</td>
<td>0.03A&lt;f_b&gt; 0.03A&lt;f_c&gt;</td>
<td>RC-3</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>57d_b</td>
<td>none</td>
<td>none</td>
<td>0.03A&lt;f_b&gt; 0.03A&lt;f_c&gt;</td>
<td>RC-3</td>
</tr>
<tr>
<td>Jin, 1995</td>
<td>C1</td>
<td>0.025</td>
<td>0.0017</td>
<td>0.18A&lt;f_b&gt;</td>
<td>RC-3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>0.025</td>
<td>0.0017</td>
<td>0.18A&lt;f_b&gt;</td>
<td>RC-3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C3</td>
<td>0.025</td>
<td>0.0017</td>
<td>0.18A&lt;f_b&gt;</td>
<td>RC-3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C4</td>
<td>0.025</td>
<td>0.0017</td>
<td>0.18A&lt;f_b&gt;</td>
<td>RC-3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C5</td>
<td>0.025</td>
<td>0.0017</td>
<td>0.18A&lt;f_b&gt;</td>
<td>RC-3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>R1</td>
<td>0.025</td>
<td>0.0017</td>
<td>0.18A&lt;f_b&gt;</td>
<td>RC-3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>R2</td>
<td>0.025</td>
<td>0.0017</td>
<td>0.18A&lt;f_b&gt;</td>
<td>RC-3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>R3</td>
<td>0.025</td>
<td>0.0017</td>
<td>0.18A&lt;f_b&gt;</td>
<td>RC-3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>R4</td>
<td>0.025</td>
<td>0.0017</td>
<td>0.18A&lt;f_b&gt;</td>
<td>RC-3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>R5</td>
<td>0.025</td>
<td>0.0017</td>
<td>0.18A&lt;f_b&gt;</td>
<td>RC-3</td>
<td></td>
</tr>
<tr>
<td>Researcher</td>
<td>ID</td>
<td>Specimen details</td>
<td>loading</td>
<td>retrofit details</td>
<td>capacity</td>
<td>ductility</td>
</tr>
<tr>
<td>------------</td>
<td>-----</td>
<td>------------------</td>
<td>---------</td>
<td>------------------</td>
<td>----------</td>
<td>-----------</td>
</tr>
<tr>
<td></td>
<td></td>
<td>d x b mm</td>
<td>$\rho_s$</td>
<td>$\rho_v$</td>
<td>splice</td>
<td>a/d ratio</td>
</tr>
<tr>
<td>Jin, 1995</td>
<td>C1R</td>
<td>305</td>
<td>0.025</td>
<td>0.0017</td>
<td>20d_p</td>
<td>$0.18A_{p}f_c^{*}$</td>
</tr>
<tr>
<td></td>
<td>C4R</td>
<td>dia.</td>
<td>none</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>R1R</td>
<td>368 x 241</td>
<td>0.027</td>
<td>0.0013</td>
<td>20d_p</td>
<td>$0.14A_{p}f_c^{*}$</td>
</tr>
<tr>
<td></td>
<td>R3R</td>
<td></td>
<td>0.055</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Osada et al.</td>
<td>1</td>
<td>300 x 450</td>
<td>0.024</td>
<td>0.0006</td>
<td>none</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
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</tr>
<tr>
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<td></td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>300 x 900</td>
<td>0.018</td>
<td>0.0003</td>
<td>none</td>
<td></td>
</tr>
<tr>
<td></td>
<td>7</td>
<td></td>
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Table 2.7 (continued) Experimental investigations of reinforced concrete columns with FRP jacket retrofits.

<table>
<thead>
<tr>
<th>Researcher</th>
<th>ID</th>
<th>Specimen details</th>
<th>loading</th>
<th>retrofit details</th>
<th>capacity</th>
<th>ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>d x b mm</td>
<td>$\rho_s$</td>
<td>$\rho_v$</td>
<td>splice</td>
<td>a/d ratio</td>
</tr>
<tr>
<td>Stanton et al., 1996</td>
<td>1</td>
<td>390 x 275</td>
<td>0.030</td>
<td>0.0030</td>
<td>none</td>
<td>1.54</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>275 x 275</td>
<td>0.010</td>
<td>0.0007</td>
<td>none</td>
<td>7.64</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>275 x 275</td>
<td></td>
<td></td>
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<td></td>
</tr>
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<td>4</td>
<td>275 x 275</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yamanaka et al. 1996</td>
<td>C1</td>
<td>390 x 390</td>
<td>0.030</td>
<td>0.0030</td>
<td>none</td>
<td>1.54</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>275 x 275</td>
<td></td>
<td></td>
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Table 2.8 Experimental investigations of steel and CFRP column retrofits conducted at UCSD (Seible, Hegemier, Priestley and Innamorato, 1995).

<table>
<thead>
<tr>
<th>Specimen details</th>
<th>a/d</th>
<th>retrofit details</th>
<th>mode of failure</th>
<th>capacity</th>
<th>ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>d x b</td>
<td>ρ_ε</td>
<td>ρ_v</td>
<td>splice</td>
<td>m_max</td>
<td>μ_90</td>
</tr>
<tr>
<td>610 (mm) dia.</td>
<td></td>
<td></td>
<td>none</td>
<td></td>
<td></td>
</tr>
<tr>
<td>610 x 0.025</td>
<td>0.010</td>
<td>0.0017</td>
<td>1.5</td>
<td>none - as built</td>
<td>F/BB</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4 - 0.5 - 4 mm, CFRP full height</td>
<td>F</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>none - as built</td>
<td>V</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>steel jacket</td>
<td>F</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.5 - 0.4 - 1.5 mm full height</td>
<td>F</td>
</tr>
<tr>
<td>610 x 0.025</td>
<td>0.025</td>
<td>0.0013</td>
<td>2.0</td>
<td>none - as built</td>
<td>LS</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>steel jacket</td>
<td>SBR</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4 - 0.5 mm 1.75d high</td>
<td>LS</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5.5-0.5 mm 1.75d high</td>
<td>SBR</td>
</tr>
<tr>
<td>610 x 405 (mm)</td>
<td>0.025</td>
<td>0.0013</td>
<td>2.0</td>
<td>none - as built</td>
<td>V</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>oval steel jacket</td>
<td>F/BR</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2-0.5-2 mm, full height</td>
<td>F</td>
</tr>
<tr>
<td>730 x 490 (mm)</td>
<td>0.050</td>
<td>0.0010</td>
<td>6.0</td>
<td>none - as built</td>
<td>BB</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>oval steel jacket</td>
<td>V</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>10-5 mm 1.88d high</td>
<td>JR</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>none - as built</td>
<td>LS</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>oval steel jacket</td>
<td>SBR</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7.5 - 0.5 mm grouted oval CFRP 1.4 d high</td>
<td>SBR</td>
</tr>
</tbody>
</table>

CFRP: M10E-AS4D12K Prepreg - f_{ult} = 1725 MPa; E = 142 GPa; ε_{ult} = 1.15%
0.06A_{f_c} applied axial load - reversed cyclic lateral loading with 3 cycles per level.
Table 2.9 Experimental investigations of steel and GFRP column retrofits conducted at UCSD (Priestley, Seible and Fyfe, 1992).

<table>
<thead>
<tr>
<th>Specimen details</th>
<th>a/d</th>
<th>retrofit details</th>
<th>mode of failure</th>
<th>capacity</th>
<th>ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>d x b (mm)</td>
<td>ρ_v</td>
<td>splice</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>610 dia.</td>
<td>0.0017</td>
<td>20d&lt;sub&gt;b&lt;/sub&gt; (0.6d)</td>
<td>6.0</td>
<td>none - as built</td>
<td>LS</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>steel jacket</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.4 mm; 1.7 MPa; 2d high</td>
<td>F</td>
<td>1.40</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.2 mm; 0.7 MPa; 2d high</td>
<td>F/BS</td>
<td>1.46</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.8 mm @ 90° and 0.6 mm @ 90°; 1.4 MPa</td>
<td>F/BS</td>
<td>1.33</td>
</tr>
<tr>
<td>610 x 405</td>
<td>0.0013</td>
<td>none</td>
<td>2.0</td>
<td>none - as built</td>
<td>V</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>steel jacket</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.3 mm; 0.7 MPa; d high</td>
<td>F</td>
<td>1.22</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.3 mm; 0.7 MPa; d high</td>
<td>F</td>
<td>1.28</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.3 mm; 0.7 MPa; d high</td>
<td>F</td>
<td>1.28</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.3 mm; 0.7 MPa; d high</td>
<td>F</td>
<td>1.28</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.3 mm; 0.7 MPa; d high</td>
<td>F</td>
<td>1.28</td>
</tr>
</tbody>
</table>

GFRP: E-Glass fabric by Knytex - active confinement provided by epoxy grout injection
\( \rho_v = 0.025 - 0.06A_{c, v} \) applied axial load - reversed cyclic lateral loading with 3 cycles per level

modes of failure: F: flexural hinging; V: shear; LS: lap splice; BS: lap splice bond slip; BR: bar rupture
BB: longitudinal bar buckling; SBR: starter bar rupture; JR: jacket rupture

50
poorly confined hinge region

large tie spacing resulting in inadequate shear capacity

short, poorly confined lap splice located in high moment region

(a)

poor tie details resulting in loss of confinement and longitudinal bar support after spalling occurs

(b)

loss of longitudinal bar support due to large tie spacing

(c)

Figure 2.1 Potentially inadequate reinforcing column details.
Figure 2.2 Uniaxial stress-strain relationships for concrete having constant confining pressure.
Figure 2.3 Geometry of confined region of core concrete.
linear relationship between $s_r$ and $f_s$

proposed relationship between $\varepsilon_r$ and $\varepsilon_s$ (Poisson effect)

stress-strain relationship for confining steel $\varepsilon_r$, $\varepsilon_s$

Figure 2.4 Schematic representation of stress-strain relationship model for concrete having passive confinement (adapted from Madas and Elnashai, 1992).
Figure 2.5 Effect of specimen shape on stress-strain response of confined concrete.
CHAPTER 3
CURRENT DESIGN GUIDELINES FOR FRP JACKET RETROFIT

This chapter reviews current design guidelines for FRP jacket retrofit of non-ductile reinforced concrete columns. Design guidelines for jackets installed using a continuous fiber wrapping system are provided by Sieble and Innamorato (1995). Sieble and Innamorato state that these design guidelines can be applied to other FRP jacket systems, if appropriate reduction factors are defined for durability, non-uniformity in lay-up for non-automated systems, non-continuous fibers or jacket joints in the hoop direction, and for systems where curing is under ambient conditions rather than in a controlled environment.

The design guidelines are organized into four sections. Section 3.1 presents guidelines for designing FRP jackets to retrofit against axial flexural failure of non-ductile reinforced concrete columns. Section 3.2 presents guidelines for retrofit against shear failure. Section 3.3 presents guidelines for retrofit against lap splice failures. Finally, section 3.4 presents guidelines for jacket lengths.

3.1 Retrofit Against Axial Flexural Failure

To retrofit against axial flexural failure, the FRP jacket is designed to provide the confinement needed to produce a ductile flexural hinge in the column. This section outlines current confinement theory proposed by Mander et al. (1998), and shows how this theory is applied to the design of steel jackets and FRP jackets.

3.1.1 Theory

To develop the required ultimate curvature required by a seismic retrofit, the flexural hinge region must be able to develop the corresponding ultimate concrete compressive strain. The FRP jacket is designed to provide the confinement needed to reach the required ultimate
compressive strain without failure of the concrete. Seible and Innamorato recommend the use of
design guidelines based on a strain balancing theory presented in Mander et al. (1988) and

Mander (1984) and Mander et al. (1988) use an energy balance method to calculate the
ultimate concrete compressive strain that can be achieved by confined concrete. This ultimate
compressive strain can be used to predict the ultimate curvature of a flexural hinge in
a reinforced concrete member. The energy balance method was devised for concrete confined by
conventional transverse reinforcing steel and later applied to steel jackets (Chai et al. 1994). This
approach assumes that the increase in energy absorbed by the confined concrete, compared to
unconfined concrete, is equal to the strain energy capacity of the transverse steel reinforcement
as it yields in tension. Thus the ultimate concrete strain at failure of the concrete corresponds to
fracture of the transverse reinforcement. The ultimate concrete strain corresponding to fracture
of transverse reinforcement is calculated by relating the strain energy capacity of the transverse
reinforcement, confined concrete, compression steel, and unconfined concrete:

\[ U_{sh} = U_{cc} + U_{sc} - U_{co} \]  \hspace{1cm} (3.1)

where:

\( U_{sh} \) = the strain energy capacity of the transverse reinforcement, per unit length of the
column;

\( U_{cc} \) = the strain energy of the core concrete confined by and contained within the
transverse reinforcement, per unit length of the column;

\( U_{sc} \) = the strain energy required to maintain yield in the longitudinal steel in compression,
per unit length of the column; and,
$U_{co} = \text{the strain energy capacity of concrete contained within the transverse}
\text{reinforcement, per unit length of the column, assuming the concrete is unconfined.}$

When the concrete member is confined by a jacket, the concrete strain energy is balanced
by not only the strain energy capacity of the transverse reinforcement, but also that of the
jacket.

Each term in the above energy equation may be expressed as an integral of the material
stress strain curve multiplied by the volume of the material within a unit length of the column.

$$U_{sh} = \rho_1 A_{cc} \int_0^{\epsilon_{sf}} f_s \, d\epsilon_s$$

(3.2)

where $\rho_1 A_{cc}$ is the volume of transverse steel within a unit length of the column, $f_s$ and $\epsilon_s$ are the
stress and strain in the transverse reinforcement, and $\epsilon_{sf}$ is the fracture strain of transverse
reinforcement.

$$U_{sc} = \rho_2 A_{cc} \int_0^{\epsilon_{cu}} f_s \, d\epsilon_c$$

(3.3)

where $\rho_2 A_{cc}$ is the volume of longitudinal steel within a unit length of the column, $f_s$ is the stress
in the longitudinal reinforcement, and $\epsilon_c$ is the longitudinal compressive strain in the concrete,
and $\epsilon_{cu}$ is the ultimate longitudinal concrete compressive strain.

$$U_{co} = A_{cc} \int_0^{\epsilon_{cu}} f_c \, d\epsilon_c$$

(3.4)

$$U_{cc} = A_{cc} \int_0^{\epsilon_{cu}} f_{cc} \, d\epsilon_{cc}$$

(3.5)
where $A_{cc}$ is the unit volume of core concrete, $f_c$ and $\epsilon_c$ are the longitudinal stress and strain for unconfined concrete, $f_{cc}$ and $\epsilon_{cc}$ are the longitudinal stress and strain for confined concrete, and $\epsilon_{sp}$ is the spalling strain of unconfined concrete.

A simplification is made in Mander et al. (1988) for the strain energy of the transverse reinforcement. The simplification is based upon reinforcing bar tension tests in which the ultimate strengths ranged from 260 MPa to 360 MPa and the ultimate strain of the bar ranged from 0.24 to 0.29. The strain energy of both the mild (260 MPa) steel and high strength (360 MPa) steel was determined to be approximately 110 MJ/m$^3$ ($\pm$ 10%) and was found to be effectively independent of bar size or yield strength. Therefore the strain energy required to fracture a volume of steel per unit length of the column is:

$$U_{sh} = \rho_s A_{cc} \frac{110 \text{ MJ}}{\text{m}^3}$$

Another simplification is made in Mander (1984) for the strain energy capacity per unit length of the column assuming unconfined concrete. This simplification is based on tests of plain concrete columns with concrete compressive strengths ranging between 28 MPa to 41 MPa. To obtain the strain energy per unit length of the column of the unconfined concrete for a complete compression failure, the experimental stress-strain curves were integrated. The results of integrating the stress-strain curves determined that the strain energy per unit volume is approximately 0.017$\sqrt{f_{c}'}$. Therefore Equation 3.4 is:

$$U_{co} = A_{cc} 0.017 \sqrt{f_{c}'} \text{MPa}$$

where $f_{c}'$ is in MPa.
With these two simplifications, the strain energy balance method provides the following equation:

\[
110 \frac{\text{MJ}}{\text{m}^3} A_t = A_{cc} \int_0^{\epsilon_{cu}} f_{cc} d\epsilon_{cc} + A_s \int_0^{\epsilon_{cu}} f_{sl} d\epsilon_{c} - A_{cc} 0.017 \sqrt{f_c} \frac{\text{MJ}}{\text{m}^3} \tag{3.8}
\]

Normalizing the above equation with respect to \(A_{cc}\) gives the following equation presented in Mander et al. (1988):

\[
110 \rho_t = \int_0^{\epsilon_{cu}} f_{c} d\epsilon_{c} + \rho_{cc} \int_0^{\epsilon_{cu}} f_{sl} d\epsilon_{c} - 0.017 \sqrt{f_c} \frac{\text{MJ}}{\text{m}^3} \tag{3.9}
\]

where: \(\rho_t\) is the volume of transverse reinforcement per unit length of the column; and, \(\rho_{cc}\) is the volume of longitudinal reinforcement per unit length of the column.

### 3.1.2 Design of Steel Jackets

Chai et al. (1994), derive a design equation for steel jackets. An axially loaded plain concrete column externally retrofit with a steel jacket is considered. The strain energy approach equates the increase in strain energy absorbed by the confined concrete compared to unconfined concrete, \(U_{cc} - U_{co}\) (per unit length of the column) to the strain energy capacity of the steel jacket, \(U_j\) (per unit length of the column). \(U_{se} = 0\) because the concrete columns did not include longitudinal steel. The strain energy balance equation is

\[
U_{cc} - U_{co} = U_j \tag{3.10}
\]

The individual terms are graphically presented in Figure 3.1. These energy expressions may again be expressed in terms of the integration of the material stress strain curves.
The integrations may be expressed in terms of material strengths.

\[
U_{cc} - U_{co} = A_{cc} \left( \int_{0}^{\epsilon_{cu}} f'_{cc} d\epsilon_{cc} - \int_{0}^{\epsilon_{sp}} f'_{c} d\epsilon_{c} \right) \quad (3.11)
\]

\[
U_{j} = A_{j} \int_{0}^{\epsilon_{j}} f_{j} d\epsilon_{j} \quad (3.12)
\]

The integrations may be expressed in terms of material strengths.

\[
U_{j} = A_{j} \gamma_{2} f_{yj} \epsilon_{jr} \quad (3.13)
\]

\[
U_{cc} - U_{co} = A_{cc} \gamma_{1} f'_{cc} \left( \epsilon_{cu} - \epsilon_{sp} \right) \quad (3.14)
\]

where: \( \gamma_{1} \) and \( \gamma_{2} \) = coefficients of integration;

\( f'_{cc} \) = confined compressive strength of concrete;

\( \epsilon_{cu} \) = ultimate concrete strain;

\( \epsilon_{sp} \) = spalling strain of unconfined concrete;

\( f_{yj} \) = yield stress for the jacket; and

\( \epsilon_{j} \) = ultimate strain for the jacket.

For a circular column, equating \( U_{cc} - U_{co} = U_{j} \) results in:

\[
\gamma_{1} f'_{cc} \left( \epsilon_{cu} - \epsilon_{sp} \right) \frac{\pi}{4} (D_{j} - 2t_{j})^{2} = \gamma_{2} f_{yj} (D_{j} - t_{j}) t_{j} \pi \quad (3.15)
\]

where \( t_{j} \) = the thickness of the steel jacket; and,

\( D_{j} \) = the diameter of the steel jacket.

The ratio of the area of the steel jacket to the area of the confined concrete can be
introduced to simplify Equation 3.15:

\[ \rho_{sj} = \frac{4t_j}{D_j - 2t_j} = \frac{4t_j (D_j - t_j)}{(D_j - 2t_j)^2} \]  \hspace{1cm} (3.16)

where: \( \rho_{sj} \) = the volume ratio of the steel jacket to the confined concrete. The values of \( \gamma_1 \) and \( \gamma_2 \) are given by Mander (1984) as:

\[ \gamma_1 = 0.8 - 1.0 \hspace{1cm} \gamma_2 = 1.35 \]

where \( \gamma_1 \) was computed for the concrete stress-strain model proposed by Mander (1984) and \( \gamma_2 \) is computed for tension tests of grade 275-380 transverse reinforcement of various bar diameters (Mander 1984). Therefore the ratio of \( \gamma_2 / \gamma_1 = 1.4\). Equation 3.15 can be solved for \( \varepsilon_{cu} \):

\[ \varepsilon_{cu} = \varepsilon_{sp} + 2.8 \varepsilon_{jr} \rho_{sj} \frac{f_{yj}}{f'_{cc}} \]  \hspace{1cm} (3.17)

Equation 3.17 can be used to compute the ultimate strain of a plain concrete column confined by a given steel jacket (Chai et al. 1994). Mander et al. (1988) state that the confined concrete compressive strength, \( f'_{cc} \), can be assumed equal to 1.5 times of the unconfined concrete compressive strength, \( f' \).

3.1.3 Design of FRP Jackets

Seible et al. (1997) proposed a procedure for determining the flexural ductility resulting from the confinement provided by an FRP jacket.

The flexural deformation capacity of a column is based on the ultimate concrete
compressive strain in the concrete, \( \varepsilon_{cu} \), from the empirical relationship presented by Chai et al. (1996):

\[
\varepsilon_{cu} = \varepsilon_{sp} + 2.8 \varepsilon_{jr} \frac{f_j}{f_{cc}'},
\]

(3.18)

where: \( \rho_j = \frac{4t_j}{D_j} \), \( t_j \) = the thickness of the jacket and \( D_j \) is the diameter of the jacket; 

\( f_{cc}' \) = the compressive strength of the confined concrete; 

\( \varepsilon_{sp} \) = the spalling strain of unconfined concrete; 

\( f_j \) = the strength of the jacket; 

\( \rho_j \) = the volumetric ratio of the jacket to the confined concrete; and 

\( \varepsilon_{jr} \) = the rupture strain of the jacket.

This equation was developed for circular columns with steel jacket retrofits under pure axial load. For square columns Seible et al. (1997) recommend doubling the thickness of the jacket in order to resist any stress concentration resulting from the shape factor. Therefore Equation 3.18 becomes:

\[
\varepsilon_{cu} = \varepsilon_{sp} + 1.4 \varepsilon_{jr} \frac{f_j}{f_{cc}'},
\]

(3.19)

The depth of the compression zone in the column, \( c_u \), is calculated from the moment curvature analysis. The resulting ultimate curvature is computed:

\[
\phi_u = \frac{\varepsilon_{cu}}{c_u}
\]

(3.20)
The curvature ductility is:

\[ \mu_{\phi} = \frac{\phi_u}{\phi_y} \]  \hspace{1cm} \text{(3.21)}

The curvature ductility can then be related to the member ductility through the following equation presented in Priestley and Park (1987) as:

\[ \mu_{\Delta} = 1 + 3 \left( \mu_{\phi} - 1 \right) \frac{L_p}{L} \left( 1 - 0.5 \frac{L_p}{L} \right) \]  \hspace{1cm} \text{(3.22)}

where: \( L = \) the length of the column in single curvature

\( L_p = \) the plastic hinge length approximated in Priestley and Park (1987) as:

\[ L_p = 0.08 L + 0.022 f_{sy} d_b \]  \hspace{1cm} \text{(3.23)}

Alternatively, if the required ultimate concrete strain is known, the required jacket thickness may be determined from Equation 3.17 assuming that the diameter of the jacket is the length of the column in the loading direction, \( D_j = D \):

\[ t_j = \frac{\rho_j D}{4} = 0.09 \frac{D (\epsilon_{cu} - \epsilon_{u}) f_{cc}'}{f_j \epsilon_{jr}} \]  \hspace{1cm} \text{(3.24)}

### 3.2 Retrofit Against Shear Failure

Sieble et al. (1997) use the shear strength model proposed by Priestley et al. (1994) to describe the shear strength of reinforced concrete columns. For a reinforced concrete column, the
model assumes that there are three shear strength components.

\[ V_n = V_c + V_s + V_p \]  

(3.25)

where: 
\( V_n \) = the nominal shear strength;
\( V_c \) = the concrete contribution provided primarily in the form of aggregate interlock;
\( V_s \) = the transverse reinforcing steel contribution; and
\( V_p \) = the axial load contribution.

For a reinforced concrete column retrofit with a steel or FRP jacket, the shear strength provided by the jacket is included. Seible et al. (1997) recommend the use of:

\[ V_n = V_c + V_s + V_p + V_j \]  

(3.26)

\[ \frac{V_n}{\phi_v} \leq V_n = V_c + V_s + V_p + V_j \]  

(3.27)

where: \( V_j \) is the contribution of the jacket.

The nominal shear strength, governed by the four shear strength components, must exceed the shear demand with an appropriate strength reduction factor, \( \phi_v \).

**Concrete Contribution** - \( V_c \) depends on aggregate interlock, which depends on the width of the cracks in the column. In a flexural hinge region, crack widths increase with increasing flexural ductility demand. The flexural ductility demand is related to the column displacement ductility level \( \mu_D \). Priestley et al. (1994) use the following equation to express the effect of flexural ductility on \( V_c \):
\[ V_c = k \sqrt{f_c} 0.8 A_g \]  

(3.28)

where: \( k \) = a strength reduction factor based on the column displacement ductility, \( \mu_\Delta \);

\( A_g \) = the gross area of the column cross section.

\[ k = 0.3 \quad \text{for} \quad \mu_\Delta < 2 \]

\[ k = 0 \quad \mu_\Delta > 4; \quad \text{low transverse reinforcement ratio} \]

\[ k = 0.1 \quad \text{for} \quad \mu_\Delta > 4; \quad \text{transverse reinforcement ratio} \]

The value of \( k \) varies linearly from 0.3 to 0.1 between \( \mu_\Delta = 2 \) to 4 for moderate transverse reinforcement ratios, and linearly from 0.3 to 0 for low transverse reinforcement ratios. The strength reduction factor, \( k \), is based on the displacement ductility, \( \mu_\Delta \), and the concrete contribution is \( V_c^i \). Outside of the flexural hinge region, a strength reduction factor of \( k = 0.3 \) is assigned, because the flexural ductility demand is zero. The concrete contribution to shear strength outside of the flexural hinge region is defined as \( V_c^o \).

**Transverse Steel Contribution** - The shear strength provided by the transverse reinforcing steel, \( V_s \), is as follows:

\[ V_s = \frac{\pi}{2} \frac{A_t f_{ty}}{s} D' \cot \theta \quad \text{(circular)} \]  

(3.29 (a))
\[ V_s = \frac{n A_t f_y D'}{s} \cot \theta \]  
(rectangular) \hspace{1cm} (3.29 \text{ (b)})

where:  
- \( A_t \) = the area of one leg of the transverse reinforcement;  
- \( n \) = number of legs of transverse reinforcement in the loading direction;  
- \( f_y \) = the yield strength of the transverse reinforcement;  
- \( s \) = the spacing of the transverse reinforcement or the spiral pitch;  
- \( \theta \) = the angle of the principal compression strut, as shown in Figure 3.2; and  
- \( D' \) = the core column dimension in the loading direction from center to center of the transverse reinforcement.

A conservative assumption for \( \theta \) is 45°, so \( \cot \theta = 1 \).

**Axial Load Contribution** - The shear strength provided by the axial load is as follows:

\[ V_p = P \tan \alpha \] \hspace{1cm} (3.30)

where:  
- \( P \) = axial load in the column; and  
- \( \alpha \) = the inclination of the compression strut with the vertical column axis.

\[ \tan \alpha = \frac{D-c}{2L} \] for single curvature
\[ \tan \alpha = \frac{D - c}{L} \] for double curvature

where: \( c \) = the distance between the neutral axis and the extreme compression fiber at the ends of the column;
\( D \) = the column dimension in the loading direction; and
\( L \) = the clear column height, as shown in Figure 3.2.

The axial load contribution diminishes quickly when columns become tall.

The sum of the previously discussed components represents the non-retrofit nominal shear strength. Design provisions for retrofit of shear and ductile shear columns also include a shear resistance provided by the external jacket.

**Jacket Contribution** - The shear strength provided by the jacket is as follows:

\[
V_j = \frac{\pi}{2} f_j t_j D \cot \theta \quad \text{(circular)} \tag{3.31 (a)}
\]

\[
V_j = 2 f_j t_j D \cot \theta \quad \text{(rectangular)} \tag{3.31 (b)}
\]

where: \( t_j \) = the jacket fiber thickness;
\( f_j \) = the stress in the jacket; and
\( D \) = the column dimension in the loading direction.

Again \( \theta \) can be assumed to be 45°.
To maintain the contribution of concrete to the shear strength, Sieble and Innamorato suggest that the jacket strains be limited to $e_{jd} = 4000 \mu \varepsilon (0.4\%)$.

$$\frac{V_o}{\phi_v} = V_j + V_c + V_s + V_p$$ (3.32)

This strain limit should be below the strain capacity of the jacket material, but higher than the yield strain of the horizontal column reinforcement, allowing the full contribution of the transverse steel to develop. Substituting Equation 3.31 (b) for the $V_j$ shear resistance into the above equation and solving for $t_j$ gives the following design:

$$t_j = \frac{\frac{V_o}{\phi_v} - (V_c + V_s + V_p)}{2 \varepsilon_{jd} E_j D}$$ (3.33)

### 3.3 Retrofit Against Lap Splice Failure

A lap splice failure occurs when the lapped longitudinal bars debond from the concrete and separate from each other while splitting the cover concrete as shown in Figure 3.3. Transverse reinforcement may prevent the splitting of the cover and provide clamping forces to prevent debonding. A clamping force provided by a lateral pressure, $f_n$, on the splice region is required to prevent the lap splice from slipping. A simplified model developed by Priestley et al. (1996) assumes that the lap splice debonding occurs in the form of failure planes in the concrete cover and along the longitudinal column bar perimeter as shown in Figure 3.3. The failure model assumes the pull out of the concrete prisms must be restrained by clamping forces across the
debonding interface. The concept of shear friction with a friction coefficient of $\mu = 1.4$ is assumed for naturally occurring concrete cracks. For a circular column as shown in Figure 3.4, equilibrium can be formulated and $f_l$ obtained by:

\[ 2 t_j f_j = f_l D \]  

where: $t_j =$ the thickness of the jacket; 

$f_j =$ the stress in the jacket; 

$D =$ the dimension of the column in the loading direction; and 

$f_l =$ the lateral clamping pressure required to keep the lap splice reinforcement from debonding.

from Equation 3.34, the jacket thickness can be found as:

\[ t_j = \frac{D f_l}{2 f_j} \]  

(3.35)

The debonding criteria is obtained from equilibrium of the tension force in the longitudinal reinforcement and the friction force acting on the longitudinal reinforcement by the concrete:

\[ f_l = \frac{A_b 1.4 f_{sy}}{\mu \left[ \frac{P}{2n} + 2 (d_b + cc) \right] L_s} \]  

where: $A_b =$ the area of one longitudinal reinforcing bar; 

$f_{sy} =$ the yield strength of the longitudinal reinforcement; 

$p =$ the inside crack perimeter along the longitudinal column reinforcement; 

$n =$ the number of bars;
\[ \mu = 1.4 = \text{the friction coefficient of concrete;} \]
\[ d_b = \text{the bar diameter;} \]
\[ c_c = \text{the concrete cover to the longitudinal column reinforcement; and} \]
\[ L_s = \text{the lap splice length.} \]

The above equation assumes a 40% overstrength of the column reinforcement past the yield stress level \( f_{sy} \). Therefore, \( 1.4f_{sy} \) needs to be developed in the lap splice.

Seible et al. (1995) report that lap splice debonding starts at transverse strains between 1000-2000 \( \mu \varepsilon \). Therefore, an allowable transverse strain of 1000 \( \mu \varepsilon \) is recommended:

\[ f_j = E_j \varepsilon_j = E_j 0.001 \quad (3.37) \]

For columns with transverse reinforcement provided by spirals, the effect of the transverse reinforcement confining force is determined through equilibrium and presented by:

\[ f_h = \frac{0.002 A_t E_s}{D_s} \quad (3.38) \]

where: \( f_h \) = the clamping force provided by the transverse reinforcement;
\( A_t \) = the area of the hoop or spiral reinforcement;
\( s \) = the center to center spacing of the transverse reinforcement
\( E_s \) = the steel modulus of elasticity; and
\( D \) = the column diameter, used to approximate the spiral diameter.

From the above relationships and the limiting strain of 1000\( \mu \varepsilon \), the jacket thickness may be obtained from the following relationship:
\[ t_j = 500 D \frac{(f_i - f_h)}{E_j} \]  

(3.39)

where: \( t_j \) = the thickness of the jacket;  
\( D \) = the column dimension in the loading direction;  
\( f_h \) = the horizontal stress level provided by the existing transverse reinforcement;  
\( E_j \) = the modulus of elasticity of the jacket; and  
\( f_i \) = the lateral clamping pressure over the lap splice described below.

Since lateral confinement pressure, \( f_i \), can be high, Seible et al. (1995) do not recommend the use of rectangular column jackets.

### 3.4 Jacket Height

Retrofit against the three failure mechanisms, as discussed in the previous sections, will require different jacket designs for different regions of the non-ductile reinforced concrete column. The lengths of these regions are defined as:  
\( L_s \) = lap splice length;  
\( L_{e1} \) = length of primary confinement region for flexural hinge;  
\( L_{e2} \) = length of secondary confinement region adjacent to flexural hinge;  
\( L_{v1} \) = length of region retrofit against shear failure inside the flexural hinge zone; and  
\( L_{v0} \) = length of region retrofit against shear failure outside the flexural hinge region by Seible et al. (1995). These regions are shown on Figure 3.4.

The length requirements reported by Seible et al. (1995) for the flexural jacket are as follows:

\[ L_{e1} \geq \frac{L}{8} \quad \text{and} \quad 0.5 \, D \]  

(3.40)
and are measured from the point of maximum moment.

Seible et al. (1995) recommend that the length of the primary confinement region required

$$L_{c2} \geq \frac{L}{8} \quad \text{and} \quad 0.5 \, D$$

(3.41)

for the jacket thickness $t_j$ extend beyond the expected plastic hinge region. They recommend that a reduced jacket thickness of 0.5 $t_j$ is to be extended for a distance $L_{c2}$ above $L_{c1}$. The stiffness of the confining material (jacket and transverse) reinforcement is gradually decreased along the column length by providing this smaller jacket thickness in the $L_{c2}$ region.

To avoid shear failure in or near a flexural hinge, the length of regions retrofit for shear are as follows:

$$L_v^i = 1.5 \, D$$

(3.42)

and $L_v^o$ is the rest of the column (outside of the flexural hinge region) subjected to the same shear force demand (Seible et al. 1995).

In order to prevent lap splice slipping, Seible et al. (1995) recommend that the length of a jacket for lap splice retrofit should be:

$$L_s \geq \text{lap splice length}$$

(3.43)
Figure 3.1 Strain energy capacity per unit length of core concrete for various materials.
Figure 3.2 Shear capacity due to applied axial load (Priestley et al. 1994).
Figure 3.3 Lap splice failure model.

Figure 3.4 Lateral clamping force developed by the jacket retrofit.

Figure 3.5 Required jacket height parameters.

\[ L_{c1} > (0.5 \ D, L/8) \]
\[ L_{c2} > (0.5 \ D, L/8) \]
\[ L_{v} > 1.5 \ D \]
\[ L_{v}^{*} = L - L_{v}^{i} \]
\[ L_{s} > \text{Lap Length} \]
CHAPTER 4

TEST SPECIMEN DESIGN, FABRICATION, RETROFIT, TEST PROCEDURE, AND INSTRUMENTATION

This chapter describes a program of tests designed to investigate the seismic behavior of non-ductile reinforced concrete columns retrofit with FRP jackets. Previous research (e.g., Priestley et al. 1992, and Kobayashi et al. 1995) has demonstrated that FRP jackets can improve the seismic behavior of non-ductile reinforced concrete bridge piers with circular cross sections. Most reinforced concrete columns in buildings, however, are square or rectangular. Furthermore, building columns may have different reinforcing details than bridge piers. The test specimens described in this chapter represent typical non-ductile reinforced concrete building columns.

The test matrix includes two non-retrofit specimens and five retrofit specimens. The specimens are further divided into specimens designed and tested to fail in either the axial flexural failure mode or the ductile shear failure mode. The test matrix is shown in Table 4.1.

Section 4.1 discusses the test specimen design. Section 4.2 summarizes the test specimen fabrication. Section 4.3 discusses test specimen material properties. Section 4.4 provides a general description of the carbon fiber tow sheet material properties. Section 4.5 summarizes the retrofit design. Section 4.6 discusses the retrofit jacket application. Section 4.7 summarizes the test procedure, and Section 4.8 discusses the instrumentation.

4.1 TEST SPECIMEN DESIGN

The test specimens were based on columns included in prototype non-ductile reinforced concrete buildings designed by Kurama et al. (1996). The cross section dimensions, reinforcement, and service axial load of the test specimens are identical to columns in the 9 story
and 12 story structures designed according to the ACI 318-63 Code (Kurama et al. 1996). The
details of the prototype building are briefly summarized below.

4.1.1 Prototype Structure Details

Three prototype structures were designed. Each structure has uniform 2.44 m (8 ft) story
heights and 6.40 m (21 ft) bays. A one-way framing system is used with primary beams spanning
between columns in the direction parallel to the seismic forces considered. Secondary beams,
with a 2.13 m (7 ft) spacing, span between the main beams in the direction perpendicular to the
direction of seismic forces considered. Each structure has 5 bays in the direction of the secondary
beams and either 3 or 5 bays in the primary direction. The prototype structures are 3, 9, and 12
stories tall and are identified by the number of stories and number of bays in the direction of the
primary beams as shown in Figure 4.1. For example, the 12 story, 5 bay structure is referred to
as 12s5b. The prototype structures considered in this investigation were designed using the
Ultimate Strength Design method of ACI 318-63.

The prototype structures were designed for gravity loads only, without seismic loads.
Snyder (1995) showed that wind design loads in effect during the 1960's were not large enough
to affect the design of the 3 and 9 story prototype structures. Wind effects were not investigated
for the 12 story prototype structures. The gravity loads considered in design are given below.
Live load reduction factors were applied to floor loads when determining member design forces
(Kurama 1993).

Floor dead loads: structural weight plus 0.7 kPa (15 psf) for interior finishes;

5.3 kN/m (360 plf) for curtain wall;

Floor live loads: 2.9 kPa (60 psf) plus 1 kPa (20 psf) partition loading.
Roof dead loads:

- Structural weight plus 0.7 kPa (15 psf) for roofing;
- 5.3 kN/m (360 plf) for parapet;

Roof live loads: 1.45 kPa (30 psf).

4.1.2 Dimensions and Details of the Test Specimens

The column dimensions and details correspond to two prototype columns designed by Kurama et al. (1996), as shown in Table 4.2. These columns are 458 mm (18 in.) square sections with 8 #7 (22 mm, 0.875 in. diameter) longitudinal reinforcing bars and #3 (10 mm, 0.375 in. diameter) transverse ties spaced at 356 mm (14 in.). The transverse ties are provided with 90° bends at their anchorage. The cross section and elevations of the test specimens are shown in Figure 4.2.

The test specimens all have the same cross-section. Different failure modes are obtained by varying the moment-to-shear ratio under which the specimen is loaded. Concrete with a specified minimum compressive strength of $f'_c = 27.6$ MPa (4 ksi) and reinforcing steel with a specified minimum yield stress of $f_y = 414$ MPa (60 ksi) were assumed to design the test specimens.

The tests specimens are divided into two regions (Figure 4.2). The lower region has widely spaced transverse reinforcement, #3 (10 mm, 0.375 in. diameter) ties spaced at 356 mm (14 in.), detailed according to the prototype columns. The upper region contains added transverse reinforcement to prevent failure in the region where the load is applied. Additional diamond shaped ties were added to confine the mid-side bars, as shown in Figure 4.2. This detailing is typical of current seismic detailing. All transverse reinforcement was fastened to the longitudinal reinforcement at the appropriate locations using wire ties. In the lower region of the column, each
square tie was terminated with 90° anchorages and had a minimum extension of 95 mm (3.8 in.), equivalent to 10 tie bar diameters. The outer dimension of each side of the ties measured 381 mm.

Footing details are shown in Figure 4.3. The top and bottom of the footing contains 6 #7 (22 mm, 0.875 in. diameter) longitudinal reinforcement. Transverse reinforcement is provided around the column longitudinal reinforcing bars in the form of #3 (10 mm, 0.375 in.) ties spaced at 152 mm (6 in.). Additional transverse reinforcement in the form of #3 bars (10 mm, 0.375 in.) is provided in the footing at 152 mm (6 in.). The footings are prestressed to the strong floor of the laboratory by four (102 mm (4 in.) diameter) steel rods. A shear key was provided at the bottom of the footing to provide additional shear slip resistance as shown in Figure 4.2 and 4.3.

4.1.3 Anticipated Behavior of Tests Specimens Under Lateral Load

The test specimens are 3050 mm (10 ft) tall cantilever columns and are tested under combined axial and lateral loads. The columns are designed to accept lateral loads at heights between 1220 mm (4 ft) and 2440 mm (8 ft). The height at which the lateral load is applied to the test specimens will control their behavior.

A moment-shear-axial load interaction for the test specimens was developed to enable the lateral load height to be established. Figure 4.4 shows the moment-shear interaction for the cross-section of the column at an axial load of 1268 kN (285 kips). This axial load is equal to 22% of the gross axial load capacity \( A_z f'_c \) where \( A_z \) is the gross column area. The moment-shear interaction was determined from the plane sections analysis program RESPONSE (Collins and Mitchell 1991). The design material properties were \( f'_c = 27.6 \text{ MPa (4 ksi)} \) for concrete and 414 MPa (60 ksi) for longitudinal and transverse reinforcing steel. The “nominal” moment and shear capacities and the “probable” moment and shear capacities were also determined from the program RESPONSE and are constant values indicated by dotted lines on the moment-shear
interaction diagram. For the nominal moment capacity, $M_n$, and nominal shear capacity, $V_n$, the concrete and steel are assumed to reach a maximum stress equal to the minimum strength specified in design ($f'_c = 27.6$ MPa (4 ksi) and $f_y = 414$ MPa (60 ksi)). For the probable moment capacity, $M_p$, and probable shear capacity, $V_p$, the steel strength is assumed to be 1.25 times the minimum specified yield stress, $f_y$, of the reinforcing steel.

The shear resistance at a displacement ductility $\mu_A = 2.3$, corresponding to 1% drift of the prototype structure as shown in Table 2.1 (Wu 1995), and at other ductilities was determined using the shear strength model proposed by Priestley et al. (1994) described in Section 3.2.2. For this calculation, the shear resistance provided by the axial load, $P$, was assumed to be zero. The shear resistance provided by the transverse reinforcing steel, $V_s$, is calculated assuming the stress in the steel is the minimum specified yield stress. The shear resistance of the transverse reinforcing steel, $V_s$, was calculated assuming the steel strength was the minimum specified yield stress, $f_y$. $V_s$ was assumed to be zero. The shear resistance of the transverse steel was calculated using Equation 3.29(b):

$$V_s = \frac{n A_t f_y D'}{s} \cot \theta \quad \text{(rectangular)} \quad (3.29(b))$$

where: $n =$ the number of plies of carbon tow sheet;

$f_y =$ the yield stress of the transverse reinforcing steel;

$D' =$ the core column dimension in the loading direction; and,

$s =$ the spacing between the transverse reinforcement.

Assuming that the angle of the diagonal compression strut, $\theta = 45^\circ$, $\cot \theta = 1$, the above equation computes $V_s$ as:
The concrete shear resistance, \( V_c \), considers degradation of aggregate interlock with increased flexural displacement ductility. The concrete shear resistance is defined by Equation 3.28:

\[
V_c = k \sqrt{f_c'} 0.8 A_g
\]  

where: \( k \) = the strength reduction factor based on \( \mu_\Delta \);

\( f_c' \) = the concrete material strength; and,

\( A_g \) = the gross area of the cross section.

Three values of \( k \) were considered assuming the column has a low transverse reinforcement ratio (Priestley et al. 1994): (1) the value at \( \mu_\Delta = 2.3 \) from Wu (1995) which is \( k = 0.255 \); (2) the value of \( k \) at \( \mu_\Delta = 3.0 \), which is \( k = 0.15 \); and, (3) the value of \( k \) at \( \mu_\Delta \geq 4.0 \) which is \( k = 0 \). In the first case the total shear resistance is \( V_{t,23} = V_s + V_c = 341 \) kN. In the second case, the shear resistance is \( V_{t,3} = 195 \) kN. In the third case, \( V_{t,4} = V_s = 63 \) kN. These values of shear strength are shown on Figure 4.5. For comparison, the shear strength, \( V_{ACI} \) was calculated using the provisions of ACI 318-95 and is shown on Figure 4.5.

In Figure 4.4, any line passing from the origin represents the monotonic lateral load history of a cantilever column with a height equal to the inverse slope of the line. In the test arrangement, the specimens are tested as cantilever columns, with the height of the applied lateral load equal to \( h_1 \). There exists a range of moment-to-shear ratios corresponding to a range of

\[
V_s = \frac{142 \text{ mm}^2 (414 \text{ MPa}) (381 \text{ mm})}{356 \text{ mm}} = 63 \text{ kN}
\]
lateral load heights for which the failure mechanism is not defined by either an axial flexural failure mode or a shear failure mode. The failure mode that occurs in this shaded region is a ductile shear failure mode. A ductile shear failure occurs when the column reaches the nominal moment capacity, $M_n$, and forms a plastic hinge. Flexural cracks develop and degrade the aggregate interlock mechanism of concrete shear resistance. Eventually the column fails in shear.

The width of the shaded ductile shear region is defined by two lines. The top line of the shaded region defines the load height below which the shear resistance $V_p$ would be reached before flexural yielding could begin at $M_n$. In this case, a flexural hinge would not form and the failure mode would be considered brittle shear. The bottom line of the shaded region defines the lateral load height above which the residual shear strength, $V_r$, is sufficient to maintain a plastic hinge with a moment capacity of $M_p$. In particular, $V_{r,3}$ is used, assuming that the plastic hinge will be maintained up to a displacement ductility of $\mu_\delta = 3$. This level of ductility is larger than expected for the non-retrofit test specimens. Between these two lines, flexural yielding is likely to occur before the shear strength is reached and the failure mode is expected to be ductile shear. A load height of $h_i = 2.44$ m (96 in.), located in the flexure critical region, was chosen for the axial flexural failure mode test specimens. A load height of $h_i = 1.52$ m (60 in.), located inside of the ductile shear region, was chosen for the ductile shear failure mode test specimens.

4.2 TEST SPECIMEN FABRICATION

The test specimens were cast in oiled wood forms, with a 25 mm (1 in.) chamfer strip secured at each corner of the column cross-section to provide the square columns with 45° chamfers. Reinforcing cages were constructed and placed inside the forms using chairs to provide proper alignment.
The column footings were cast first, and after 17 days the columns were cast. A cold joint was located at the column footing interface. The columns were cast in a vertical position. Concrete was placed in three equal lifts. Following each lift, the concrete was consolidated using an electric-powered submersion vibrator. After the concrete placement, wet burlap and plastic sheeting covered the column tops for the first seven days. After 17 days, the forms were stripped and the columns were allowed to air-cure. A total of eight columns were cast, in two separate pours. For each pour, standard 152 x 305 mm (6 x 12 in.) concrete cylinders were prepared according to ASTM C 31-90. The cylinders were also covered by wet burlap and plastic sheeting for seven days, after which time they were stripped from their molds and air-cured.

4.3 TEST SPECIMEN MATERIAL PROPERTIES

4.3.1 Concrete Properties

A 27.6 MPa (4000 psi) concrete was specified with 20 mm (3/4 in.) maximum aggregate size, Type I cement, 3-5% air entrainment, and 102 mm (4 in.) slump. The compressive strength of the concrete was determined from tests performed on standard 152 x 305 mm (6 x 12 in.) concrete cylinders. The cylinders were tested in a 2660 kN (600 kip) capacity universal testing machine according to ASTM C 90-86. Concrete cylinder test results from the column concrete are reported in Table 4.3. A 34.5 MPa (5000 psi) with 20 mm (3/4 in.) maximum aggregate, Type I cement, 3-5% air entrainment, and 102 (4 in.) slump was specified for the footing.

4.3.2 Steel Reinforcement Properties

The reinforcing steel was ASTM A615, Grade 60 steel. All column longitudinal steel was from the same lot of material. All transverse steel was taken from the same lot of material, different than that of the longitudinal steel. Tension tests were conducted on the longitudinal and transverse reinforcing steel according to ASTM A 615-87 to determine the yield stress, $f_y$, and
ultimate strength, $f_u$. The tests were performed in a 2660 kN (600 kip) universal testing machine. Table 4.4 summarizes the results of the longitudinal and transverse reinforcement tests. Footing reinforcing steel was also specified as ASTM A615, Grade 60 steel.

4.4 CARBON FIBER Tow sheet PROPERTIES

The unidirectional carbon fiber tow sheet used for the FRP jackets in this investigation was provided by Tonen Corporation of Japan (hereafter referred to as Tonen), (Forca FTS-C1-30 carbon fiber tow sheet). The tow sheet is provided as a 500 mm (19.5 in.) wide roll of carbon tow sheet with fibers aligned in the longitudinal direction of the roll. The fiber strands are held in alignment by a stitched grid of fibers on one side of the sheet. This side of the sheet is lightly bonded to a sheet of paper which serves to protect the fibers during handling. A sample of the material is shown in Figure 4.5. Table 4.5 summarizes the material properties of the FORCA FTS-C1-30 carbon fiber tow sheet that were reported by Tonen. The manufacturer reported material rupture strain is $\varepsilon_{ju} = 15000 \mu \varepsilon$ (1.5%). In situ jacket rupture strains recorded during axial load tests (Kestner et al. 1997) were $\varepsilon_{ju} = 9000 \mu \varepsilon$. Demers (1994) also reported in situ jacket rupture strains lower than the material rupture strain reported by the manufacturer. The carbon fiber tow sheet used in this investigation is the same as the material used by Kestner et al. (1997). An in situ jacket rupture strain of $\varepsilon_{ju} = 9000 \mu \varepsilon$ (0.9%) was used for retrofit design.

4.5 RETROFIT DESIGN

As discussed earlier, five of the non-ductile reinforced concrete column test specimens were retrofit with FRP jackets comprised of one or more plies of carbon fiber tow sheet impregnated with an epoxy resin.
4.5.1 Axial Flexural Specimens

As shown in Table 4.1, the number of plies of carbon fiber tow sheet in the FRP jackets varied between the three retrofit axial flexural test specimens. The design of these jackets is discussed in this section.

Required Ultimate Curvature

The required curvature ductility was chosen to be 4.3, corresponding to 1% drift of the prototype structure as shown in Table 2.1 (Wu 1995). This level of curvature ductility would be required in the 3S5B prototype structure for Zone 2A (Kurama et al. 1996), typical of the eastern and central United States. To determine the required ultimate curvature, the yield curvature was obtained and multiplied by the curvature ductility. The yield curvature of the non-retrofit column cross-section was computed from analysis of a fiber section model using the DRAIN-2DX computer program (Prakash et al. 1993). The fiber model descriptizes the section into fibers which may be assigned individual material properties. Figure 4.6 shows the descretization of the cross-section into layers. Each layer has one fiber for the unconfined concrete. Additional fibers are added to model the longitudinal reinforcement. Each fiber is located at the centroid of the layer and is assigned a material property and cross sectional area. A section analysis of the fiber model is conducted with strain compatibility enforced across the section. For the non-retrofit specimen, the stress-strain model for unconfined concrete proposed by Popovics (1973) and modified by Thorenfeldt et al. (1987) and Collins and Porasz (1989) was used. An elastic-perfectly plastic stress-strain model was used for the longitudinal reinforcing steel. The DRAIN-2DX fiber beam-column element uses a multi-linear relationship for the stress-strain behavior of concrete and steel. The multi-linear relationship is defined by as many as five discrete stress-strain pairs. Table 4.6 gives the stress and strain values used in this analysis. Figure 4.6 shows the cross section and the
stress-strain curves. The analysis of the fiber model gives a yield curvature of:

\[ \Phi_y = 17 \times 10^{-6} \frac{1}{\text{mm}} \]

From this yield curvature and the required curvature ductility of 4.3, the ultimate curvature of the section was determined:

\[ \Phi_u = \mu \Phi_y = 4.3 \times 17 \times 10^{-6} \frac{1}{\text{mm}} = 73 \times 10^{-6} \frac{1}{\text{mm}} \]

**Design of Jacket Retrofit Based on a Fiber Section Analysis**

Fiber section analyses of columns with proposed FRP jackets with 2, 3, 4, 6, and 9 plies of carbon fiber tow sheet were carried out using DRAIN-2DX (Prakash et al. 1993). The fiber model includes the geometry of the cross-section, and multi-linear stress-strain curves for steel and for unconfined and confined concrete.

Figure 4.7 shows the cross-section geometry. Areas for confined concrete and unconfined concrete were determined using the model proposed by Restrepo and DeVino (1996). The model assumes that the boundary between confined and unconfined concrete is a segment of a semi-circle (Figure 4.7(a)). Each layer of the cross-section in the fiber model includes a fiber for confined concrete and a fiber for unconfined concrete. Additional fibers were used to model the longitudinal reinforcing bars (not shown). Each fiber is located at the centroid of the layer and is assigned a material property and cross sectional area.

The stress-strain behavior of unconfined concrete, confined concrete, and reinforcing steel is included in the model. The behavior of the confined concrete is derived using the variably
confined concrete model (VCCM) proposed (Harries et al. 1997) and described in Section 2.4.5. The VCCM is based on a circular cross-section. To use this model for a square cross-section, a diameter, $D_d$ equal to the diagonal dimension of the square section, $D_d = D\sqrt{2}$ was used. The diameter, $D_d$, was selected assuming that the confining pressure developed across the diagonals of the square cross-section is equivalent to the confining pressure developed on a circular cross-section with a diameter equal to this diagonal dimension. The VCCM provides a stress-strain curve for concrete confined by an FRP jacket. Key parameters for the model include jacket stiffness, jacket strength, number of plies in the FRP jacket, and the limiting concrete dilation ratio, $\epsilon_{cu}$, which is a function of jacket stiffness. For the 2, 3, 4, 6, and 9 ply jacket designs, the limiting dilation ratio was selected using test data developed by Kestner et al. (1997) and plotted in Figure 4.8 versus normalized jacket stiffness. A normalized jacket stiffness was computed based on the number of plies of carbon tow sheet in each jacket design and was plotted within the test data in Figure 4.8. The numbers inside circles in the figure correspond to the number of plies of carbon fiber tow sheet for each jacket retrofit considered. The limiting dilation ratio was then determined for each jacket design. A stress-strain curve for confined concrete was generated for each jacket design using the VCCM. A typical stress-strain curve is shown in Figure 4.7 (d). A multi-linear relationship for this stress-strain curve was defined by the data values provided in Table 4.7. The last data point of the multi-linear stress strain curve represents the point at which the jacket ruptures. This point corresponds to the ultimate concrete compressive strain which is related to the transverse strain through the limiting dilation ratio, $\epsilon_{cu}$ (Kestner et al. 1997). At jacket rupture, the transverse strain and the jacket rupture strain are equal to:

$$\epsilon_{jr} = \eta \epsilon_{cu}$$

(4.1)
The ultimate concrete compressive strain, $\varepsilon_{\text{con}}$, is the longitudinal strain in the concrete at which the jacket will rupture. For the confined concrete stress-strain curve, the last point of the multi-linear relationship is the point at which the strain is $\varepsilon_{\text{con}}$. At this last data point, the stress-strain curve in the model reaches a plateau. Since jacket failure occurs at this point, concrete strains on the stress-strain curve plateau are beyond the failure strain and indicate failure of the cross-section.

The stress-strain curve used for unconfined concrete is the same as that used in the non-retrofit column. However, a residual stress is included to account for the residual strength of the concrete that crushes and within the semi-circular sections along the faces of the column and within the jacket shown in Figure 4.7 (b). An elastic-perfectly plastic stress-strain model was used for the longitudinal reinforcing steel. The stress-strain relationships for concrete are shown in Figure 4.7. Stress and strain values are given in Table 4.7.

Results of the fiber section analysis are presented in the form of a curvature, $\phi$, vs. concrete compressive strain at the extreme fiber, $\varepsilon_{\text{c,ef}}$, diagram shown in Figure 4.9. An in situ jacket strain of $\varepsilon_{\text{jr}} = 9000 \mu\varepsilon$ (0.9%) reported by Kestner et al. (1997) is used to define failure by jacket rupture. Using the limiting dilation ratios chosen from Figure 4.8, an ultimate concrete compressive strain, $\varepsilon_{\text{up}}$, is determined from Equation 4.1:

$$
\varepsilon_{\text{cu}} = \frac{\varepsilon_{\text{jr}}}{\eta}
$$

(4.2)

Using different values of $\varepsilon_{\text{up}}$ which depend on the limiting dilation ratio, $\eta$, the ultimate curvature shown in from Figure 4.9 when the concrete compressive strain at the extreme fiber $\varepsilon_{\text{c,ef}}$ reaches
$\varepsilon_{cu}$, indicating jacket rupture. If the ultimate curvature is higher than the required ultimate curvature, $\phi_u = 73 \times 10^{-6} \text{ 1/mm}$, the jacket design is considered to be acceptable.

Due to the gradient in compressive strain across the cross section, the dilation strains of the section are not constant. As a result, the ultimate concrete compressive strain, $\varepsilon_{cu}$, may be exceeded at the extreme concrete compression fiber (Watson et al. 1994) without rupture of the jacket. To account for the strain gradient, a second situation was considered in which the jacket was assumed to rupture when the concrete compressive strain at the centroid of the triangular concrete compressive strain distribution reaches $\varepsilon_{cu}$. If jacket rupture occurs when $\varepsilon_{cu}$ is reached at a distance equal to one third of the depth of the neutral axis (NA) from the extreme compression fiber, the corresponding compressive strain at the extreme fiber is:

$$\varepsilon_{c,ef} = \frac{3}{2} \varepsilon_{cu} \quad (4.3)$$

Figure 4.10 shows the ultimate curvature when $\varepsilon_{c,ef}$ reaches $1.5 \varepsilon_{cu}$. If the ultimate curvature exceeds the required ultimate curvature, $\phi_u = 73 \times 10^{-6} \text{ 1/mm}$, the jacket design is considered acceptable. A summary of jacket designs is presented in Table 4.8. Jackets with 2, 3, 4, 6, and 9 plies were investigated. For acceptable jacket designs, the ultimate curvature value is shaded in the table. Note that some jacket designs are acceptable only if the jacket is assumed to rupture when the concrete strain at one third of the NA depth from the extreme compression fiber reaches $\varepsilon_{cu}$. A 2 ply jacket (Specimen F3) is adequate under this assumption. A 4 ply jacket (Specimen F2) is adequate without this assumption.

Also provided in Table 4.8 are the maximum moment and the yield curvature values. The maximum moment is not significantly affected by the jacket retrofit. The maximum increase in
moment is 10%. The yield curvature is increased by the jacket retrofit, which is considered to be an additional benefit.

**Design of Jacket Retrofit Based on Seible et al. (1997)**

Section 3.1 reviewed jacket design guidelines developed by Seible et al. (1997). Due to variability of jacket thickness due to the hand lay-up techniques, the jacket designs are developed in this section using FRP material strength and stiffness properties given in units of force per unit length perpendicular to the principle direction of the fibers per ply, kN/(mm · ply). This allows jacket designs to be based on the stiffness and strength which can be developed in each hand-layed ply of tow sheet and resin without concern for the thickness of the ply.

The force per unit length perpendicular to the direction of the fiber is the jacket thickness times the jacket stress:

\[ t_j f_j = n \bar{f}_j = n \bar{E}_j e_j \]  

(4.4)

where:  
- \( n \) = the number of discrete plies of FRP material;  
- \( t_j \) = the jacket thickness;  
- \( f_j \) = the jacket stress;  
- \( \bar{f}_j \) = the force per length per ply;  
- \( \bar{E}_j \) = the stiffness per length per ply; and,  
- \( e_j \) = the jacket strain.

Equation 3.24 may be rewritten using Equation 4.4 and assuming the jacket stress is at the rupture stresses (\( \bar{f}_j = \bar{f}_{jr} \)) and substituting \( n \bar{f}_{jr} \) for \( t_j \bar{f}_{jr} \):

\[ t_j \bar{f}_{jr} = n \bar{f}_{jr} = n \bar{E}_j e_{jr} = \frac{0.09 \bar{D} f_{cc}}{e_{jr}} (e_{cu} - \epsilon_u) \]  

(4.5)
At rupture of the jacket, the strain is equal to $\epsilon_{jr}$, thus the force per unit length of jacket is

$$n \overline{F}_{jr} = n \bar{E}_j \epsilon_{jr}$$

The design approach outlined in Section 3.1, based on Seible et al. (1997), requires an ultimate concrete compressive strain to determine the number of plies in the jacket. The required ultimate concrete compressive strain was found from the section analysis results given in the previous section. The required ultimate curvature, $\phi_u = 73 \times 10^{-6}$ rad/mm, was determined previously, and this value was taken into the section analysis results as shown in Figure 4.11. The ultimate concrete compressive strain corresponding to the required ultimate curvature was determined for each jacket design. Then, the required number of jacket plies was determined using Equation 4.6 and the carbon tow sheet material properties presented in Section 4.4. Equation 4.5 was modified recalling that Seible et al. (1997) recommend doubling the number of plies for square or rectangular columns as follows:

$$n \overline{E}_j \epsilon_{jr} = \frac{2 \times 0.09 D f_{cc}^e}{\epsilon_{jr}} (\epsilon_{cu} - \epsilon_u)$$  \hspace{1cm} (4.6)

The required number of jacket plies was determined from Equation 4.6 with $\epsilon_{jr} = 9000 \mu e$, $\overline{E}_j = 38.0$ kN/(mm·ply), $D = 458$ mm, $f_{cc}^e = 1.5 f'_c$ and $\epsilon_{cu}$ determined from the section analysis results in Figure 4.11. If the required number of jacket plies is greater than or equal to the number of plies used in the jacket design and corresponding section analysis, the jacket is considered to be acceptable. Table 4.9 summarizes the results.

Based on the results in Table 4.4, Specimen F1 was chosen to have 6 plies of carbon tow sheet.
As discussed in Section 3.4, the required height of the jacket is the larger of the two values:

\[
\frac{L}{8} = \frac{2438}{8} = 203 \text{ mm}
\]

\[
\frac{D}{2} = \frac{458}{2} = 229 \text{ mm}
\]

However, the required length of the jacket should extend above the top of the plastic hinge zone. Thus, the height of the jacket should be greater than the plastic hinge length determined from Equation 3.23:

\[
L_p = 0.08L + 0.022 \frac{1}{\text{MPa}} f_{sy} d_b
\]

\[
L_p = 0.08 \times (2438 \text{ mm}) + 0.022 \times \frac{1}{\text{MPa}} \times (414 \text{ MPa}) \times (22 \text{ mm}) = 395 \text{ mm}
\]

Using this relationship, the required jacket height is 395 mm. Since the carbon tow sheet came in a 500 mm width, 500 mm was used rather than cutting the carbon tow sheet.

4.5.2 Ductile Shear Specimens

The shear strength of a non-ductile reinforced concrete column specimen includes contributions from the concrete, \( V_c \), the steel, \( V_s \), and the axial load, \( V_p \), components of shear.
resistance as discussed in Section 3.2. To design the FRP jacket retrofit against the ductile shear failure mode, the shear strength of a non-retrofit specimen, discussed in Section 6.3.2, was reviewed. The shear strength of a non-retrofit specimen was calculated using ACI 318-95, and the shear strength model proposed by Priestley et al. (1994). Figure 4.4 shows the calculated shear strengths. The design of the jackets for the ductile shear specimens was based on the results from the model proposed by Priestley et al. (1994) for low transverse reinforcement, where at large displacement ductilities \( \mu_d \geq 4 \), the concrete shear resistance is zero.

The shear demand used for the jacket design was calculated from the moment vs. shear interaction diagram presented in Figure 4.4. For a column of 1.52 m (60 in.), the monotonic response of the column is shown in Figure 4.4 by a line passing through the origin with a slope equal to the inverse of the lateral load height. The goal of the retrofit is to obtain the plastic hinge moment and maintain the shear capacity required to resist the shear demand corresponding to this plastic hinge moment. Therefore, the point lying on the 1.52 m (60 in.) line corresponding to a moment value of \( M = M_p \) was used to determine the shear demand:

\[
V_o = \frac{M_p}{h} = \frac{430 \text{ kN m}}{1.52 \text{ m}} = 283 \text{ kN}
\]

This point is shown on Figure 4.4 by an “●”.

The jacket design approach proposed by Seible and Innamorato (1995) seeks to prevent degradation of the aggregate interlock mechanism of concrete shear resistance by limiting the dilation strain of the column (Section 3.2). Seible and Innamorato (1995) recommend limiting the dilation strain to \( \varepsilon_{jd} = 4000 \mu \varepsilon \) (0.4%) to maintain aggregate interlock. Using this approach, Equation 3.33 was developed.
Due to variability of jacket thickness due to the hand lay-up techniques, the jacket designs
are developed using FRP material strength and stiffness properties given in units of force per unit
length perpendicular to the principle direction of the fibers per ply, kN/(mm \cdot ply). This allows
jacket designs to be based on the stiffness and strength which can be developed in each hand-
layed ply of tow sheet and resin without concern for the thickness of the ply.

The force per unit length perpendicular to the direction of the fiber is the jacket thickness
times the jacket stress $t_j f_j = n T_j = n E_j$ as discussed in Section 4.5.1. Equation 3.33 may be
rewritten substituting $n T_j$ for $t_j f_j$ as:

$$n = \frac{V_c}{\phi_v} - \left( V_c + V_s + V_p \right)$$

where: $D = \text{the dimension of the column in the loading direction};$

$\phi_v = \text{the shear resistance factor};$ and,

$\epsilon_{jd} = \text{the permissible dilation strain of concrete}.$

The concrete shear resistance, $V_c$, at displacement ductilities $\mu_d = 2.3, \mu_d = 3, \text{and } \mu_d = 4$ were taken from the results used in section 4.1.3. For $\mu_d = 2.3, V_{c,2.3} = 278 \text{ kN}$, for $\mu_d = 3, V_{c,3} = 132 \text{ kN}$, and for $\mu_d = 4, V_{c,4} = 0 \text{ kN}.$

The number of plies required to retrofit a ductile shear specimen was determined using
Equation 4.7, with $\epsilon_{jd} = 4000 \mu, E_j = 38.0 \text{ kN (mm\cdotply)},$ and $D = 458 \text{ mm}.$ When $V_c = 278 \text{ kN}$ for $\mu_d = 2.3$ is used in Equation 4.7, a retrofit is not required because $\phi_v (V_c + V_s) = 290 \text{ kN} > V_o = 283 \text{ kN}.$ When $V_c = 132 \text{ kN}$ for $\mu_d = 3$ is used in Equation 4.7, a one ply jacket is required. When $V_c = 0$ for $\mu_d = 4$ is used in 4.7, a two ply jacket is required. In Section 4.5.1, the minimum
jacket designed for the axial flexural failure mode was two plies. Therefore, the number of plies required to retrofit a ductile shear specimen is governed by retrofit against the axial flexural failure mode. Specimen DV1 was designed with two plies of carbon tow sheet.

Since the concrete shear resistance, \(V_c\), degrades to zero for a column with low transverse reinforcement, a jacket design approach based on maintaining the aggregate interlock mechanism by limiting column dilation is not appropriate. If the dilation limit of \(\varepsilon_d = 4000 \mu \varepsilon\) is lifted and the concrete shear resistance is not considered, the increase in total shear resistance as a result of the shear resistance provided by the jacket at jacket rupture can be considered. The shear resistance provided by the jacket, \(V_{jr}\), is:

\[
V_j = 2n \bar{E}_j e_{jr} D \cot \theta
\]  

(4.8)

For DV1, the shear resistance of the jacket at jacket rupture is \(V_{jr} = 626\) kN, from Equation 4.8 with \(n = 2\), \(e_{jr} = 9000 \mu \varepsilon\), \(\bar{E}_j = 38\) kN/(mm•ply), \(D = 458\) mm, and \(\cot \theta = 1\). The sum of \(V_s\) and \(V_{jr} = 689\) kN, which is significantly larger than \(V_o = 283\) kN. Therefore, Specimen DV1 is expected to behave as a retrofit axial flexural specimen, because the shear strength demand is greatly exceeded by the shear strength. Therefore, the jacket does more than limit dilation and maintain the shear resistance of the concrete. The jacket increases the ultimate shear resistance of the column by an additional shear resistance component.

The jacket retrofit of Specimen DV2 was designed with four plies of carbon tow sheet. This jacket was designed to provide increased ductility in the axial flexural failure mode, recognizing that the retrofit ductile shear specimens are flexure critical. As a result, Specimens DV2 and F2, with four plies of carbon tow sheet are directly comparable.
4.6 JACKET RETROFIT APPLICATION

The carbon FRP jackets were applied using a three step procedure: (1) prepare the concrete surface; (2) apply primer to the surface; and (3) apply the jacket in a series of plies.

Concrete surface preparation began by grinding the rough surfaces on the column smooth. The corners of the column were ground to an approximate 50 mm (2 in.) radius, and surface voids were patched with hydrostone and allowed to cure. The excess hydrostone was ground smooth using sandpaper. The hydrostone was allowed to harden for at least 24 hours before the surface was primed.

A two part epoxy-resin prime coat was mixed with a 2:1 ratio of resin to hardener. The weight of the primer per unit surface area was 0.25 kg/m\(^2\) (0.05 lb/ft\(^2\)). The primer coat was cured for a minimum of 20 hours before the jacket was applied.

The carbon fiber tow sheet for each ply was cut with the adhesive backing paper intact to a 1905 mm (75 in.) length. This length was sufficient to wrap once around the column and provide an additional 102 mm (4 in.) lap which was recommended by the manufacturer. After the tow sheet for all plies was cut, a three part application process began as shown in Figure 4.12.

A two part epoxy resin mixed with a 2:1 ratio of resin to hardener was used to bond the tow sheet to the column. The required weight of resin per unit surface area was 0.81 kg/m\(^2\) (0.16 lb/ft\(^2\)) per ply of jacket. Seventy percent of the epoxy resin (by weight) was mixed and applied directly to the primed surface using solid small nap rollers. The tow sheet was then positioned with the unidirectional fiber in contact with the column surface and the stitched grid of fibers and adhesive backing paper on the outside. One person wrapped the tow sheet around the column pressing down on the it to reduce the amount of entrapped air, while another person aligned the it. Once the initial portion of the tow sheet was in place, the adhesive paper backing was removed.
from the tow sheet. The tow sheet was rolled in the direction of application using a grooved 19 mm (0.75 in.) aluminum roller to impregnate the carbon fibers with the resin and remove any entrapped air. After the ply was rolled, the lap was secured using additional resin between the two pieces of tow sheet. The remaining thirty percent of the epoxy resin was mixed and applied to the outside tow sheet approximately one half hour after the initial seventy percent of the epoxy had been applied. This resin was then rolled into the tow sheet using the grooved aluminum roller. Once the carbon fiber tow sheet was well impregnated with resin and the entrapped air was removed, the resin was allowed to cure for 90 minutes before the next ply was applied.

The application procedure was repeated for each ply and the laps were placed on different sides of the column until the total number of plies were in place. Figure 4.13 shows the dimensions and number of plies of the jackets used to retrofit the test specimens for the axial flexural and ductile shear specimens.

4.7 TEST PROCEDURE

To ensure a uniform bearing between the column base and the strong floor of the test lab, the column base was grouted in place with a layer of hydrostone. The tests specimens were tested under combined axial and lateral loads. A constant axial load of 1267 kN (285 kips), representing expected service level loads was applied. This level of axial load is equal to 22% of the column gross axial capacity ($A_g f'_c$).

The test-set up is shown in Figure 4.14. All specimens were tested using a 1068 kN, 952 mm (240 kip, 38 in.) stroke actuator to provide lateral loads and two 1334 kN (300 kip) rams to apply axial load. The axial load was applied under load control. Reversed cyclic lateral load was applied under load control up to the yield displacement. After yield, displacement control was used for the lateral load application. A series of elastic cycles were conducted, which were
followed by yield cycles. The column yield displacement was defined by a flattening in the load vs. deflection curve after yield of the longitudinal reinforcement had occurred. Cycles were then continued at several multiples of the yield displacement until failure occurred. The load history is shown in Figure 4.15 for each specimen.

A data acquisition system including a power supply, signal conditioners, A/D boards, and a computer was used to scan the data during the test. A schematic of the data acquisition system is shown in Figure 4.16. As the axial load was applied to the test specimens, the data acquisition system recorded the data at increments of the total axial load. After the axial load was on the column, the data acquisition system recorded the data at increments of lateral displacement. Additionally, the computer displayed twenty channels of data to allow test data to be monitored during testing.

4.8 INSTRUMENTATION

4.8.1 Non-Retrofit Specimens

The instrumentation used for the non-retrofit specimens is shown schematically in Figure 4.17. Lateral load was measured using a load cell attached to the lateral load actuator and axial load was measured using load cells attached to the two axial load rams. Two linear variable differential transducers (LVDTs) were used to measure lateral displacement. Two additional LVDTs were used to measure column displacement near the footing. Rotation meters were used to measure rotation of the column and rotation of the footing. In addition, linear potentiometers were used to measure relative displacements in the hinge region of the column. From these linear potentiometers, curvatures could be calculated. Linear potentiometers were also used on one side of the column to measure relative displacements that were used to calculate curvature and shear distortion. Strain gages were placed on the longitudinal and transverse reinforcing bars.
4.8.2 Retrofit Specimens

The instrumentation used for the non-retrofit specimens was also used for the retrofit specimens. Additional strain gages and rotation meters were provided on the jacketed specimens. Figure 4.18 shows the positioning of these additional strain gages and rotation meters for the various tests.
Table 4.1 Test matrix.

<table>
<thead>
<tr>
<th>Axial Flexural Specimens</th>
<th>Ductile Shear Specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen</td>
<td>number of plies</td>
</tr>
<tr>
<td>F0</td>
<td>0</td>
</tr>
<tr>
<td>F1</td>
<td>6</td>
</tr>
<tr>
<td>F2</td>
<td>4</td>
</tr>
<tr>
<td>F3</td>
<td>2</td>
</tr>
</tbody>
</table>

Table 4.2 Column details and axial service loads for prototype structures.

<table>
<thead>
<tr>
<th>Prototype Story</th>
<th>Interior Columns</th>
<th>Exterior Columns</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>d mm reinforcing details</td>
<td>axial service load kN</td>
</tr>
<tr>
<td></td>
<td>long. trans.</td>
<td></td>
</tr>
<tr>
<td>12s 9s 3s</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12 9 3</td>
<td>356 4 #7 #3 @ 356</td>
<td>267</td>
</tr>
<tr>
<td>11 8 2</td>
<td>356 4 #7 #3 @ 356</td>
<td>543</td>
</tr>
<tr>
<td>10 7 1</td>
<td>406 8 #6 #3 @ 305</td>
<td>796</td>
</tr>
<tr>
<td>9 6</td>
<td>458 8 #6 #3 @ 305</td>
<td>1036</td>
</tr>
<tr>
<td>8 5</td>
<td>458 8 #7 #3 @ 356</td>
<td>1285</td>
</tr>
<tr>
<td>7 4</td>
<td>508 8 #7 #3 @ 356</td>
<td>1552</td>
</tr>
<tr>
<td>6 3</td>
<td>559 8 #8 #3 @ 406</td>
<td>1824</td>
</tr>
<tr>
<td>5 2</td>
<td>559 12 #8 #3 @ 406</td>
<td>2095</td>
</tr>
<tr>
<td>4 1</td>
<td>610 12 #8 #3 @ 406</td>
<td>2375</td>
</tr>
<tr>
<td>3</td>
<td>660 12 #8 #3 @ 406</td>
<td>2655</td>
</tr>
<tr>
<td>2</td>
<td>660 12 #9 #3 @ 458</td>
<td>2936</td>
</tr>
<tr>
<td>1</td>
<td>660 12 #10 #3 @ 458</td>
<td>3216</td>
</tr>
</tbody>
</table>
### Table 4.3 Concrete material properties.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>28 day strength, MPa (psi)</th>
<th>Test day</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Age (days)</td>
<td>Strength, MPa (psi)</td>
</tr>
<tr>
<td>F0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F1</td>
<td>24.8</td>
<td>24.6 (3561)</td>
</tr>
<tr>
<td>F2</td>
<td>231</td>
<td>22.7 (3288)</td>
</tr>
<tr>
<td>F3</td>
<td>253</td>
<td>not tested</td>
</tr>
<tr>
<td>DV0</td>
<td>353</td>
<td>24.1 (3499)</td>
</tr>
<tr>
<td>DV1</td>
<td>240</td>
<td>24.9 (3610)</td>
</tr>
<tr>
<td>DV2</td>
<td>305</td>
<td>24.6 (3565)</td>
</tr>
</tbody>
</table>

### Table 4.4 Reinforcing bar material properties.

<table>
<thead>
<tr>
<th>Steel</th>
<th>$f_y$, MPa (ksi)</th>
<th>$f_u$, MPa (ksi)</th>
<th>$\varepsilon_u$, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>#7 longitudinal steel</td>
<td>460 (66.7)</td>
<td>680 (98.7)</td>
<td>16%</td>
</tr>
<tr>
<td>#3 transverse steel</td>
<td>438 (63.6)</td>
<td>714 (103.6)</td>
<td>11%</td>
</tr>
</tbody>
</table>

### Table 4.5 Carbon fiber reinforced polymer (CFRP) material properties.

<table>
<thead>
<tr>
<th>CFRP material data</th>
<th>$T_f$, N/(mm•ply) (lbs/(mm•ply))</th>
<th>$E_f$, kN/(mm•ply) (kips/(in•ply))</th>
<th>$\varepsilon_f$, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manufacturer Data</td>
<td>580 (3310)</td>
<td>38.0 (220)</td>
<td>1.5</td>
</tr>
<tr>
<td>C1-30 Tow Sheet</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 4.6 Stress-strain pairs used in DRAIN-2DX fiber section analysis.

<table>
<thead>
<tr>
<th>Material</th>
<th>Stress-Strain Pairs</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MPa</td>
</tr>
<tr>
<td></td>
<td>mm/mm</td>
</tr>
<tr>
<td>unconfined f&lt;sub&gt;residual&lt;/sub&gt; = 5 or 15 MPa</td>
<td>21.5</td>
</tr>
<tr>
<td></td>
<td>0.00110</td>
</tr>
<tr>
<td>reinforcing steel</td>
<td>460</td>
</tr>
<tr>
<td></td>
<td>0.00230</td>
</tr>
</tbody>
</table>

* indicates data point not required for multi-linear relationship

Table 4.7 Stress-strain pairs generated from VCCM used in DRAIN-2DX fiber section analysis.

<table>
<thead>
<tr>
<th>Material</th>
<th>Dilation ratio</th>
<th>Stress-Strain Pairs</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>η</td>
<td>MPa</td>
</tr>
<tr>
<td></td>
<td></td>
<td>mm/mm</td>
</tr>
<tr>
<td>confined 2 plies CFRP</td>
<td>0.7</td>
<td>18.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.00100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>24.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.00175</td>
</tr>
<tr>
<td></td>
<td></td>
<td>28.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.00420</td>
</tr>
<tr>
<td></td>
<td></td>
<td>34.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.01286</td>
</tr>
<tr>
<td>confined 3 plies CFRP</td>
<td>0.6</td>
<td>18.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.00092</td>
</tr>
<tr>
<td></td>
<td></td>
<td>24.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.00158</td>
</tr>
<tr>
<td></td>
<td></td>
<td>28.0</td>
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<tr>
<td></td>
<td></td>
<td>0.00267</td>
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<tr>
<td></td>
<td></td>
<td>31.5</td>
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<td></td>
<td></td>
<td>0.00525</td>
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<td></td>
<td></td>
<td>40.0</td>
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<td></td>
<td></td>
<td>0.01500</td>
</tr>
<tr>
<td>confined 4 plies CFRP</td>
<td>0.56</td>
<td>18.0</td>
</tr>
<tr>
<td></td>
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</tr>
<tr>
<td></td>
<td></td>
<td>24.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.00175</td>
</tr>
<tr>
<td></td>
<td></td>
<td>30.6</td>
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<tr>
<td></td>
<td></td>
<td>0.00875</td>
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<td>44.6</td>
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<td></td>
<td>0.01610</td>
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<tr>
<td>confined 6 plies CFRP</td>
<td>0.5</td>
<td>18.0</td>
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<td>52.7</td>
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<td></td>
<td>0.01800</td>
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<tr>
<td>confined 9 plies CFRP</td>
<td>0.4</td>
<td>16.7</td>
</tr>
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<td>0.00079</td>
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<td></td>
<td></td>
<td>0.01040</td>
</tr>
<tr>
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<td></td>
<td>61.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.02230</td>
</tr>
</tbody>
</table>

* indicates data point not required for multi-linear relationship
Table 4.8 Design of Jacket Retrofit Based on a Fiber Section Analysis

<table>
<thead>
<tr>
<th>Plies</th>
<th>η</th>
<th>( f_{\text{residual}} ) MPa</th>
<th>( \varepsilon_{\text{ov}} ) με</th>
<th>( \phi_y ) ( 10^{-6}/\text{mm} )</th>
<th>( \varepsilon_{\text{c,ef}} = \varepsilon_{\text{ov}} )</th>
<th>( \phi_u ) ( 10^{-6}/\text{mm} )</th>
<th>( M_{\text{max}} ) kNm</th>
<th>( \phi_u ) ( 10^{-6}/\text{mm} )</th>
<th>( M_{\text{max}} ) kNm</th>
<th>( \phi_u ) ( 10^{-6}/\text{mm} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>N/A</td>
<td>0</td>
<td>8100</td>
<td>16.58</td>
<td>376.2</td>
<td>35</td>
<td>376.2</td>
<td>35</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 (F3)</td>
<td>0.70</td>
<td>5</td>
<td>12900</td>
<td>17.80</td>
<td>385.7</td>
<td>59</td>
<td>387.8</td>
<td>78</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>15</td>
<td>12900</td>
<td>20.12</td>
<td>398.4</td>
<td>70</td>
<td>402.6</td>
<td>102</td>
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<td></td>
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<td>3</td>
<td>0.60</td>
<td>5</td>
<td>15000</td>
<td>18.53</td>
<td>390.5</td>
<td>64</td>
<td>390.5</td>
<td>108</td>
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</tr>
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<td></td>
<td>15</td>
<td>15000</td>
<td>21.26</td>
<td>403.9</td>
<td>77</td>
<td>407.5</td>
<td>127</td>
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<td></td>
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<tr>
<td>4 (F2)</td>
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<td>5</td>
<td>16100</td>
<td>18.98</td>
<td>392.6</td>
<td>81</td>
<td>395.3</td>
<td>130</td>
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<td>16100</td>
<td>21.89</td>
<td>408.4</td>
<td>90</td>
<td>415.6</td>
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<td>6</td>
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<td>18000</td>
<td>19.40</td>
<td>394.5</td>
<td>104</td>
<td>403.1</td>
<td>158</td>
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<tr>
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<td></td>
<td>15</td>
<td>18000</td>
<td>22.92</td>
<td>414.6</td>
<td>112</td>
<td>423.2</td>
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<td>9</td>
<td>0.40</td>
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<td>22500</td>
<td>21.21</td>
<td>405.0</td>
<td>135</td>
<td>413.6</td>
<td>214</td>
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<td></td>
<td>15</td>
<td>22500</td>
<td>23.35</td>
<td>422.6</td>
<td>148</td>
<td>429.2</td>
<td>234</td>
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</tbody>
</table>
Table 4.9 Design of jacket retrofit based on Seible et al. (1997).

<table>
<thead>
<tr>
<th>Number of plies</th>
<th>Unconfined concrete residual stress, MPa</th>
<th>Ultimate compressive concrete strain, με</th>
<th>Required number of plies</th>
<th>Adequacy of design</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>5</td>
<td>17900</td>
<td>8.2</td>
<td>NO</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>14900</td>
<td>6.4</td>
<td>NO</td>
</tr>
<tr>
<td>3</td>
<td>5</td>
<td>16000</td>
<td>7.1</td>
<td>NO</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>14200</td>
<td>6.0</td>
<td>NO</td>
</tr>
<tr>
<td>4</td>
<td>5</td>
<td>14200</td>
<td>6.0</td>
<td>NO</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>13700</td>
<td>5.7</td>
<td>NO</td>
</tr>
<tr>
<td>6 (F1)</td>
<td>5</td>
<td>13900</td>
<td>5.8</td>
<td>YES</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>13500</td>
<td>5.6</td>
<td>YES</td>
</tr>
<tr>
<td>9</td>
<td>5</td>
<td>13400</td>
<td>5.5</td>
<td>YES</td>
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<td></td>
<td>15</td>
<td>13200</td>
<td>5.4</td>
<td>YES</td>
</tr>
</tbody>
</table>

Table 4.10 Design of ductile shear column jacket.

<table>
<thead>
<tr>
<th>Displacement Ductility</th>
<th>V_o = 283 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>V_s</td>
</tr>
<tr>
<td>0</td>
<td>63 kN</td>
</tr>
<tr>
<td>2.3 (Wu, 1995)</td>
<td>63 kN</td>
</tr>
<tr>
<td>3</td>
<td>63 kN</td>
</tr>
<tr>
<td>4</td>
<td>63 kN</td>
</tr>
</tbody>
</table>
Figure 4.1 Prototype frame structures (Kurama et al. 1996).
Figure 4.2 Column reinforcement details.
4" sleeves

36"

24"

2 1/2"

12"

14"

square shear key
(shaded area grouted)

6 - #7 longitudinal bars top and bottom
#3 ties at 6 inches horizontal and vertical
1 1/2" cover provided all around
1 " cover provided above shear key (3 1/2" above base)

Figure 4.3 Footing dimensions and reinforcing details.
458 mm square column
8 #7 longitudinal bars
#3 transverse ties at 356 mm
axial load = 1268 kN
\( f' = 27.6 \text{ MPa} \)
\( f_y = 414 \text{ MPa} \)

\[ V_{ACI} = 289 \text{ kN} \]
\[ V_n = 264 \text{ kN} \]
\[ V_{r,s} = 195 \text{ kN} \]
\[ V_{r,t} = 63 \text{ kN} \]

Figure 4.4 Moment shear interaction relationship for test specimens.
Figure 4.4 Moment shear interaction relationship for test specimens.
(a) uni-directional carbon tow sheet

(a) stress-strain properties

Figure 4.5 Carbon fiber tow sheet.

\( E_r = 235 \text{ GPa (33,000 ksi)} \)
\( f_r = 3550 \text{ MPa (505 ksi)} \)
\( \varepsilon_r = 0.015 \)
INTENTIONAL SECOND EXPOSURE

(a) uni-directional carbon tow sheet

\[ E_i = 235 \text{ GPa (33,000 ksi)} \]
\[ f_p = 3550 \text{ MPa (505 ksi)} \]
\[ \varepsilon_p = 0.015 \]

(a) stress-strain properties

Figure 4.5 Carbon fiber tow sheet.
Figure 4.6 Non-retrofit cross-section geometry and material stress-strain relationships for fiber section analysis.
Figure 4.7 Retrofit cross-section geometry and material stress-strain relationships for fiber section analysis.
Figure 4.8 Variation of limiting dilation ratios with normalized jacket stiffness (Kestner et al. 1997).
Figure 4.9 Fiber section analysis results using $e_{cu} = e_{cf}/\eta$
Figure 4.10 Fiber section analysis results using $\varepsilon_{\text{ef}} = \frac{3}{2} (\varepsilon_{\text{cu}})$.
Figure 4.11 Fiber section analysis results $\varepsilon_{c,cf}$ determined from $\Phi_a$. 

2 plies: $\varepsilon_{c,cf} = 0.0179$
3 plies: $\varepsilon_{c,cf} = 0.0160$
4 plies: $\varepsilon_{c,cf} = 0.0142$
6 plies: $\varepsilon_{c,cf} = 0.0139$
9 plies: $\varepsilon_{c,cf} = 0.0134$

$\Phi_a = 73$

Curvature (rad/mm)

Concrete strain at the extreme fiber, $\varepsilon_{ct}$

$f_{\text{midal}} = 5 \, \text{Mpa}$

Curvature (rad/mm)

Concrete strain at the extreme fiber, $\varepsilon_{ct}$

$f_{\text{midal}} = 15 \, \text{MPa}$
Figure 4.12 Carbon tow sheet jacket application.
(a) surface preparation, grinding the concrete corners

(b) applying epoxy resin after the first ply of material has been applied

(c) orienting and applying carbon tow sheet

(d) rolling carbon tow sheet into epoxy resin

Figure 4.12 Carbon tow sheet jacket application.
Figure 4.13 Retrofit column details.
axial load = 0.22A_f' £

reversed cyclic lateral load

Axial Flexural Specimens:
2440 mm (8 ft)

Ductile Shear Specimens:
1524 mm (5 ft)
Figure 4.14 Test set-up.
Figure 4.15 Load vs. displacement history.
Figure 4.15 Load vs. displacement history.
Figure 4.15 Load vs. displacement history.

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linear potentiometers

LVDTs

inclinometer

load cells

column bar strain gages

column bar strain gages

column bar strain gages

FRP jacket strain gages

FRP jacket strain gages

FRP jacket strain gages

FRP jacket strain gages

FRP jacket strain gages

FRP jacket strain gages

FRP jacket strain gages

FRP jacket strain gages

8 channel strain gage signal conditioners
Figure 4.17 (a) Column instrumentation details.
instrument pin detail
- "as built" specimens -

instrument pins affixed to exterior of jacket on jacketed specimens

linear potentiometers
west face of test specimen

Figure 4.17 (b) Column instrumentation details.
Figure 4.17 (c) Column instrumentation details.
Figure 4.18 Additional instrumentation for retrofit columns.

- Potentiometer Pin
- Strain Gage
- Clinometer
CHAPTER 5

TEST RESULTS

The results of the tests of non-retrofit and retrofit column test specimens are presented in this chapter. Two non-retrofit and five retrofit specimens were tested. The non-retrofit specimens included one specimen which failed in the axial flexural mode and one specimen which failed in the ductile shear mode. The retrofit specimens included three specimens retrofit against the axial flexural mode and two specimens retrofit against the ductile shear failure mode. Section 5.1 provides an overview of the test results. Section 5.2 provides test results for the axial flexural test specimens, and Section 5.3 provides test results for the ductile shear test specimens. Finally, Section 5.4 provides an overall response of the retrofit test specimens.

5.1 OVERVIEW OF TEST RESULTS

The orientation of the column specimens is shown in Figure 5.1. The directions North, South, East, and West are used throughout Chapter 5 and Chapter 6 to refer to specific sides of the column. The columns were subjected to fully reversed cyclic loading in the north and south direction. Each cycle began with loading in the positive (north) direction and followed by fully reversed loading in the negative (south) direction. The north and south faces at the base of the columns were subjected to alternating tension and compression bending stresses according to the direction of applied lateral load. The applied load vs. displacement response of each specimen is presented in Figure 5.2.

Tables 5.1 and 5.2 present selected results of the tests that indicate overall behavior of the column specimens. The tables include the retrofit details, height of applied lateral load, and the value of constant axial load. A summary of applied lateral load, lateral displacement, and maximum curvature at 254 mm (10 in.) from the column footing interface 178 mm (7 in.) for
Specimen DV1) is provided at several different displacement ductility levels, at the displacement ductility where the maximum lateral load occurred, and at the displacement ductility where the maximum sustainable lateral load occurred. The maximum sustainable lateral load was defined to be 80% of the lateral load carrying capacity and reflects a 20% loss in lateral load resistance due to damage. A sustainable lateral load occurs when the applied lateral load at all three cycles exceeds 80% of the lateral load carrying capacity. If a longitudinal reinforcing bar ruptured or the jacket ruptured, the displacement ductility for the rupture is also reported.

Tables 5.3 and 5.4 present selected results of the tests that show the behavior of the column jackets. The tables include retrofit details, height of applied lateral load, and the value of constant axial load. The maximum jacket strains on the compression (north or south), tension (north or south), and east faces are presented at several different displacement ductility levels, at the displacement ductility where the maximum lateral load occurred, and at the displacement ductility where the maximum sustainable lateral load occurred. Jacket strains are reported in units of microstrain, με.

The following sections (Sections 5.2 and 5.3) provide more detailed test observations and results. Loads at first cracking and yield are reported, and the failure mode of each test specimen is described. Load vs. displacement responses are presented as well as photographs and sketches of the non-retrofit and retrofit columns during testing. For each test, the locations of transverse cracks are measured from the column footing interface. The yield displacement of each specimen was based on an observed softening of the load vs. displacement graph, and from strains measured in the longitudinal reinforcing bars. The descriptions of the jacket behavior use the terms “bulging,” “buckling,” and “rippling.” These terms are defined in Figure 5.3. Bulging of the jacket is the movement of the jacket on the compression face away from the core of the concrete.
column, and occurs as a result of column dilation. This response occurs over a region of the column. The beginning and the end of the bulging region is not easily defined. Therefore, the location of the bulge is measured at the center of the bulging region. Buckling of the jacket is the formation of a discontinuity on the compression face of the column that extends away from the core of the column. Whereas bulging is a general movement of the entire jacket away from the core concrete, buckling is a localized discontinuity of the jacket. Rippling is the formation of several small buckles which occur on the corner of the column.

5.2 AXIAL FLEXURAL TEST SPECIMENS

5.2.1 Non-retrofit Specimen F0

Figure 5.2(a) presents the applied load vs. displacement response of Specimen F0. First cracking at the interface of the column and the footing was observed at a lateral load of 62 kN (14.0 kips). As the test continued, yielding was observed to occur at a lateral load of 181 kN (40.7 kips). During the cycles at the yield displacement, increased cracking and crack opening occurred on the north and south faces of the column. The maximum load of 229 kN (51.5 kips) was recorded during the first cycle to \(2\delta_y\). The peak load in subsequent cycles was less than the maximum load of 229 kN (51.5 kips). Crushing of the concrete was observed during the first cycles to \(2.5\delta_y\). Longitudinal reinforcing bar buckling occurred during the \(2.5\delta_y\) cycles and the lateral load carrying capacity could not be maintained. Figure 5.4 shows Specimen F0 at \(2.5\delta_y\).

5.2.2 Retrofit Specimen F1

Figure 5.2(b) presents the applied load vs. displacement response of Specimen F1. First cracking at the interface of the column and the footing was observed at a lateral load of 98 kN (22.0 kips). As the test continued, yielding was observed at a lateral load of 185 kN (41.6 kips). The cycles at yield produced transverse cracks at 178 mm (7 in.) from the column footing.
interface on the south face and at 171 mm (6.75 in.) from the column footing interface on the north face of the column. Following yield, opening of these two cracks and the cracks at the interface of the column and the footing accounted for most of the tensile deformation in the hinge region, although transverse cracking occurred in the jacket at locations further up the column. At the third cycle to $5\delta_y$, bulging began 50 mm (2 in.) from the column and footing interface on the north face, and 406 mm (16 in.) from the column footing interface on the south face of the column. An increase in bulging occurred in the remaining cycles. At $8\delta_y$, the test procedure had to be changed to cyclic loading only in the south direction, because the stroke limit on the actuator in the north direction was reached. The load history for Specimen F1 is shown in Figure 4.15.

During loading in the south direction in the second cycle to a displacement ductility of $10\delta_y$, a longitudinal reinforcing bar on the north face ruptured, and the 50 mm (2 in.) bulge on the north face of the column appeared to increase in size. A loss of lateral load resistance of 40 kN (9.0 kips) occurred. After three cycles to $10\delta_y$, three cycles to $12\delta_y$ were applied. No additional loss in lateral load resistance was observed. However, the test was completed without applying additional load cycles beyond $12\delta_y$ because the stroke limits on the LVDTs measuring the lateral displacement were reached.

Following the test, the jacket was removed to verify the longitudinal reinforcing bar rupture. The longitudinal reinforcing bar rupture was located at the column footing interface in the center longitudinal bar on the north face of the column. Shear cracks were not present on either the east or west face of the column. Specimen F1 is shown in Figure 5.5 at a displacement ductility of $8\delta_y$. 

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5.2.3 Retrofit Specimen F2

Figure 5.2(c) presents the applied load vs. displacement response of Specimen F2. First cracking at the interface of the column and the footing was observed at a lateral load of 85 kN (19.0 kips). As the test continued, yielding was observed at a lateral load of 169 kN (38.1 kips). The cycles at the yield displacement produced several transverse cracks in the jacket at heights of 178 mm (7 in.) and 381 mm (15 in.) on the south face of the column and 178 mm (7 in.) and 356 mm (14 in.) on the north face of the column. Following yield, openings of these cracks and the cracks at the interface of the column and the footing accounted for most of the tensile deformation in the hinge region, although transverse cracking occurred in the jacket at locations further up the column. The crack openings increased significantly as the ductility demand increased. During cycles to $3\delta_y$, bulging of the jacket began 102 mm (4 in.) from the footing on the north and south faces of the column. During cycles to $4\delta_y$, visible signs of concrete crushing was evident through the crack openings at 178 mm (7 in.). An increase in bulging occurred in the remaining cycles. At $5\delta_y$, a buckle was noted at 381 mm (15 in.) from the column footing interface on the south face.

On the second north cycle to $8\delta_y$, a longitudinal reinforcing bar ruptured on the south face of the column. The first longitudinal reinforcing bar ruptured on the north face of the column on the second south cycle to $8\delta_y$. As the test proceeded, the jacket partially ruptured on the south face of the column during the third north cycle to $8\delta_y$. The rupture of the jacket was centered at 114 mm (4.5 in.) from the column footing interface and was 64 mm (2.5 in.) in length. The jacket ruptured on the tension face of the column. This jacket rupture likely occurred due to movement of the ruptured bar on the south face of the column. The first north cycle to $10\delta_y$, produced the second longitudinal reinforcing bar rupture on the south face of the column. A loss in lateral load
resistance of 45 kN (10.0 kips) occurred as a result of these bar ruptures. More of the jacket ruptured on the south face of the column 165 mm (6.5 in.) from the column footing interface and 38 mm (1.5 in.) in length. The first south cycle to $10\delta_y$ produced second and third bar ruptures on the north face of the column. The jacket on the north side of the column remained intact. Of the eight longitudinal reinforcing bars, only three had not ruptured.

Following the test, the jacket was removed to locate the longitudinal reinforcing bar ruptures. All longitudinal reinforcing bar ruptures were located at 178 mm (7 in.) above the column footing interface, at the location of the first transverse tie. Shear cracks were not evident underneath the jacket. Figure 5.6 shows Specimen F2 at $8\delta_y$.

5.2.4 Specimen F3

Figure 5.2(d) presents the applied load vs. displacement response of Specimen F3. First cracking at the interface of the column and the footing was observed at a lateral load of 67 kN (15.0 kips). As the test continued, yielding was observed at a lateral load of 185 kN (41.6 kips). The cycles at the yield displacement produced several transverse cracks in the jacket at heights of 185 mm (7.25 in.) and 432 mm (17 in.) on the south face and 229 mm (9 in.) 413 mm (16.25 in.) on the north face of the column. After cycles to $2.5\delta_y$, opening of these cracks and the cracks at the interface of the column and footing accounted for most of the tensile deformation in the hinge region, although transverse cracking occurred in the jacket at locations further up the column. During the cycles to $2.5\delta_y$, bulging and buckling of the jacket began to occur at 127 mm (5 in.) and 229 mm (9 in.) from the footing on the north and south faces of the column, respectively. At $6\delta_y$ a new buckle occurred on the north face of the column at 343 mm (13.5 in.) and on the south face of the column at 445 (17.5 in.). A series of ripples along the corners of the column were observed. The jacket was in distress, evidenced by rupture of groups of individual
carbon fibers. Figure 5.7 shows the retrofit column one half cycle prior to failure.

During the second south cycle to $6\delta_p$, the jacket ruptured at the south east corner at 305 mm (12 in.) above the footing. The jacket rupture was 102 mm (4 in.) in length and was oriented longitudinally as shown in Figure 5.8. A loss in lateral load resistance of 18 kN (4 kips) was observed between the second and third cycles to $6\delta_p$. More of the jacket ruptured during the third cycle to $6\delta_p$.

During the first south cycle to $7\delta_p$, all three longitudinal bar buckled on the south face of the column where the jacket had ruptured. The column was brought back to zero displacement and the axial load was removed.

5.3 DUCTILE SHEAR TEST SPECIMENS

5.3.1 Specimen DV0

Figure 5.2(e) presents the applied load vs. displacement response of Specimen DV0. First cracking at the interface of the column and the footing was observed at a lateral load of 107 kN (24.0 kips). As the test continued, yielding was observed to occur at a lateral load of 277 kN (62.3 kips). During the yield cycles increased cracking and crack opening on both the north and south faces of the column occurred. The maximum load of 342 kN (77 kips) was recorded during the first cycle to $2\delta_p$. The peak load in subsequent cycles was less than the maximum load of 342 kN (77 kips). Cracking and crack opening on the east face of the column was observed during cycles to $1.5\delta_p$, $2.0\delta_p$, and $2.5\delta_p$. These cracks were inclined shear cracks which initiated at existing flexural cracks. Relative horizontal displacement due to sliding along the cracks was observed on both the north and south faces of the column. During cycles to $1.5\delta_p$, $2.0\delta_p$, and $2.5\delta_p$, the development of new flexural and flexure-shear cracks and the opening of existing flexural and flexure-shear cracks decreased and relative sliding along existing cracks and crushing in the
plastic hinge region of the column increased. Crushing of concrete on the north and south faces of the column was observed during the first cycle to \(2\delta_y\), between 51 mm (2 in.) and 127 mm (5 in.) from the column footing interface.

During the \(2\delta_y\) cycles, mechanical problems in the hydraulic system for the lateral load occurred. The hydraulic pressure turned off unexpectedly and the column drifted slowly back to zero displacement. The hydraulic system was restarted and the test continued. During the \(2.5\delta_y\) cycles the concrete crushed completely between 51 mm (2 in.) and 127 mm (5 in.) on the north face of the column. In addition, the aggregate interlock component of the concrete shear resistance was lost and a large diagonal shear crack formed, which was initiated at the column footing interface on the north face of the column and progressed to the south face at 813 mm (32 in.) from the column footing interface as shown in Figure 5.9. The column failed in a ductile shear manner. The column obtained the flexural capacity and failed by loss of the aggregate interlock component of the concrete shear resistance. Cycling was continued to observe the post failure response of the column. The longitudinal reinforcing bars buckled during the next cycle as shown in Figure 5.10 and the test was concluded.

5.3.2 Specimen DV1

Figure 5.2(f) presents the applied load vs. displacement response of Specimen DV1. First cracking at the interface of the column and the footing was observed at a lateral load of 126 kN (28.0 kips). As the test continued, yielding was observed at a lateral load of 305 kN (68.5 kips). The cycles at yield produced several transverse cracks in the jacket at heights of 178 mm (7 in.) 375 mm (14.75 in.) on the south face and at 191 mm (7.5 in.) and 419 mm (16.5 in.) on the north face of the column. Openings of these cracks and the cracks at the interface of the column and the footing accounted for most of the tensile deformation in the hinge region, although transverse
cracking occurred in the jacket at locations further up the column. During cycles to 4$\delta_y$, bulging and a small amount of buckling appeared on the north and south faces of the column at 126 mm (7.0 in.) from the column footing interface. Bulging increased during the cycles to 5$\delta_y$ and 6$\delta_y$, and buckling increased during the 6$\delta_y$ cycles. During the third north cycle to 7$\delta_y$, a longitudinal reinforcing bar ruptured on the south face of the column. A decrease in load of 53 kN (12.0 kips) was observed between the second north cycle to 7$\delta_y$ to the third north cycle to 7$\delta_y$ when the longitudinal reinforcing bar ruptured.

During the first north cycle to 8$\delta_y$, the jacket ruptured on the northeast face at 191 mm (7.5 in.) from the column footing interface and was 51 mm (2 in.) in length as shown in Figure 5.11. After one complete load cycle to 8$\delta_y$, the column was loaded to 9$\delta_y$ in the north direction. The jacket ruptured again just below the initial jacket rupture on the north face and was 51 mm (2 in.) in length.

Following the test, the jacket was removed to locate the bar rupture. The longitudinal reinforcing bar rupture was located at 178 mm (7 in.) from the column footing interface, the location of the first transverse tie. In addition, shear cracks were observed on the east and west faces of the column. Figure 5.12 shows Specimen DV1 at a displacement ductility of 6$\delta_y$.

### 5.3.3 Specimen DV2

Figure 5.2(g) presents the applied load vs. displacement response of Specimen DV2. First cracking at the interface of the column and the footing was observed at a lateral load of 98 kN (22 kips). As the test continued, yielding was observed at a lateral load of 292 kN (65.7 kips). The yield cycles produced several transverse cracks in the jacket at heights of 165 mm (6.5 in.) and 318 mm (12.5 in.) on the south face of the column and at 197 (7.75 in.) 368 mm (14.5 in.) on the north face of the column. Openings of these cracks and the cracks at the interface of the column
and the footing accounted for most of the tensile deformation in the hinge region, although transverse cracking occurred in the jacket at locations further up the column. During cycles to $4\delta_y$, bulging began 102 mm (4 in.) from the footing on the north and south faces of the column. An increase in bulging occurred in the remaining load cycles. At $5\delta_y$, a buckle occurred at 381 mm (15 in.) from the footing.

During the second north cycle to $10\delta_y$, a longitudinal reinforcing bar rupture occurred on the south face of the column. A longitudinal reinforcing bar rupture occurred on the north face of the column during the second south cycle to $10\delta_y$. A loss in lateral load resistance of 114 kN (26 kips) occurred as a result of these bar ruptures. The third cycle to $10\delta_y$ produced a rupture of a second bar on the south face. Additionally, the jacket ruptured on the south face of the column at 127 mm (5 in.) from the column footing interface, which was 52 mm (2 in.) in length. The first cycle to $12\delta_y$ produced a rupture of a third bar on the south face, and a second bar on the north face. Of the eight longitudinal reinforcing bars, only three remained.

Following the test, the jacket was removed to locate the bar rupture. All bar ruptures were located at 178 mm (7 in.) from the footing, the location of the first transverse tie. In addition, shear distress was evidenced by the formation of shear cracks on the east and west faces of the column. Figure 5.13 shows Specimen DV2 during testing.

5.4 OVERALL RESPONSE OF RETROFIT SPECIMENS

All of the retrofit specimens, with the exception of Specimen F3, failed as a result of longitudinal reinforcing bar rupture. The carbon tow sheet FRP jacket does not have sufficient bending stiffness to restrain the crushed concrete on the compression faces and buckling of the center longitudinal bar. Thus, the longitudinal reinforcing bar goes through a few cycles of buckling and straightening after the nearby cover concrete becomes damaged. The buckling and
straightening of the bar, ultimately causes the bar to rupture as a result of low cycle fatigue, as shown by the smooth rupture surface of the longitudinal reinforcing bar.

Figure 5.14 shows Specimens DV0 and DV1 following jacket removal, and the presence of shear cracks. Specimen DV2 exhibited the same shear crack pattern as that seen on Specimen DV1. After jacket removal of the axial flexural specimens, shear cracks were not observed. The shear cracks present in the retrofit ductile shear specimens indicate behavior similar to that of the non-retrofit ductile shear specimen, which failed as a result of a large diagonal shear crack as shown in Figures 5.14 and 5.9.

Investigation of the retrofit specimens after removal of the jacket showed that the concrete remained bonded to the jacket. However there was significant crushing of the concrete beneath the jacket surface. The crushed concrete beneath the jacket in the retrofit specimens appeared to continue to carry load after it was crushed, because of the confinement provided by the jacket, the crushed concrete of the specimens spalls and does not maintain load carrying capacity.
Table 5.1 (a) Observed lateral load, displacement, and curvature of axial flexural test specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>F0</th>
<th>F1</th>
<th>F2</th>
<th>F3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Retrofit details</td>
<td>non-retrofit control specimen</td>
<td>6 plies @ 500 (20)</td>
<td>4 plies @ 500 (20)</td>
<td>2 plies @ 500 (20)</td>
</tr>
<tr>
<td>Height of Applied Lateral Load, h, mm (in.)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Axial load, P, kN (kips)</td>
<td>0.25Agfc' = 1299 (292)</td>
<td>0.26Agfc' = 1339 (301)</td>
<td>0.26Agfc' = 1352 (304)</td>
<td>0.26Agfc' = 1334 (300)</td>
</tr>
<tr>
<td>Half Cycle</td>
<td>N</td>
<td>S</td>
<td>N</td>
<td>S</td>
</tr>
<tr>
<td>applied lateral load, kN (kips)</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1st</td>
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<td>-178 (-40.0)</td>
<td>185 (41.6)</td>
<td>-167 (-37.6)</td>
</tr>
<tr>
<td>3rd</td>
<td>184 (41.3)</td>
<td>-159 (-35.7)</td>
<td>170 (38.2)</td>
<td>-155 (-34.9)</td>
</tr>
<tr>
<td>lateral displacement, δy, mm (in.)</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1st</td>
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<td>-23 (-0.92)</td>
<td>23 (0.90)</td>
<td>-22 (-0.88)</td>
</tr>
<tr>
<td>3rd</td>
<td>24 (0.96)</td>
<td>-24 (-0.95)</td>
<td>23 (0.90)</td>
<td>-23 (-0.91)</td>
</tr>
<tr>
<td>maximum curvature, rad/10^6 mm (rad/10^6 in.)</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1st</td>
<td>10.1 (256)</td>
<td>-9.9 (-251)</td>
<td>7.2 (182)</td>
<td>-8.9 (-226)</td>
</tr>
<tr>
<td>3rd</td>
<td>13.0 (329)</td>
<td>-8.3 (-211)</td>
<td>6.8 (172)</td>
<td>-8.9 (-226)</td>
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Table 5.1 (b) Observed lateral load, displacement, and curvature of axial flexural test specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
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<th>F1</th>
<th>F2</th>
<th>F3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Retrofit details mm (in.)</td>
<td>non-retrofit control specimen</td>
<td>6 plies @ 500 (20)</td>
<td>4 plies @ 500 (20)</td>
<td>.2 plies @ 500 (20)</td>
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<td>2 plies @ 500-1000 (20-39)</td>
<td>1 ply @ 500-1000 (20-39)</td>
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<td>2419 (95.25)</td>
<td>2419 (95.25)</td>
<td>2413 (95.00)</td>
</tr>
<tr>
<td>Load, h, mm (in.)</td>
<td>0.25A_{r}f_y= 1299 (292)</td>
<td>0.26A_{r}f_y = 1339 (301)</td>
<td>0.26A_{r}f_y = 1352 (304)</td>
<td>0.26A_{r}f_y = 1334 (300)</td>
</tr>
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<td>Axial load, P, kN (kips)</td>
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<tr>
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<td>-202 (-45.5)</td>
<td>-202 (-45.4)</td>
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<tr>
<td></td>
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<td>213 (47.9)</td>
<td>217 (48.7)</td>
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<td>-202 (-45.4)</td>
<td>-202 (-45.4)</td>
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<tr>
<td></td>
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<td>213 (47.9)</td>
<td>217 (48.7)</td>
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<td>-202 (-45.5)</td>
<td>-202 (-45.4)</td>
<td>-202 (-45.4)</td>
</tr>
<tr>
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<td>217 (48.7)</td>
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<td>-202 (-45.4)</td>
<td>-202 (-45.4)</td>
<td>-202 (-45.4)</td>
</tr>
<tr>
<td>Applied lateral load, kN (kips)</td>
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<td>-193 (-43.4)</td>
<td>216 (48.5)</td>
<td>216 (48.5)</td>
</tr>
<tr>
<td></td>
<td>216 (48.5)</td>
<td>-203 (-45.7)</td>
<td>213 (47.9)</td>
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<td>213 (47.9)</td>
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Table 5.1 (c) Observed lateral load, displacement, and curvature of axial flexural test specimens.

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<tr>
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<td>non-retrofit</td>
<td>6 plies @ 500 (20)</td>
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<td>2 plies @ 500 (20)</td>
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<td>mm (in.)</td>
<td>control specimen</td>
<td>3 plies @ 500-1000</td>
<td>2 plies @ 500-1000</td>
<td>1 ply @ 500-1000</td>
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<tr>
<td>Height of Applied Lateral Load, h, mm (in.)</td>
<td>2369 (93.25)</td>
<td>2419 (95.25)</td>
<td>2419 (95.25)</td>
<td>2413 (95.00)</td>
</tr>
<tr>
<td>Axial load, P, kN (kips)</td>
<td>0.25A_{f'_c} = 1299 (292)</td>
<td>0.26A_{f'_c} = 1339 (301)</td>
<td>0.26A_{f'_c} = 1352 (304)</td>
<td>0.26A_{f'_c} = 1334 (300)</td>
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<th>N</th>
<th>S</th>
<th>N</th>
<th>S</th>
<th>N</th>
<th>S</th>
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<tr>
<td>1st applied lateral load, kN (kips)</td>
<td>N/A</td>
<td>N/A</td>
<td>222 (50.0)</td>
<td>-214 (-48.0)</td>
<td>217 (48.7)</td>
<td>-201 (-45.1)</td>
<td>219 (49.2)</td>
<td>-184 (-41.4)</td>
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<tr>
<td>3rd</td>
<td>N/A</td>
<td>N/A</td>
<td>230 (51.6)</td>
<td>-210 (-47.1)</td>
<td>214 (48.0)</td>
<td>-195 (-43.8)</td>
<td>203 (45.7)</td>
<td>-149 (-33.5)</td>
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<td>1st lateral displacement, ( \delta_c ), mm (in.)</td>
<td>N/A</td>
<td>N/A</td>
<td>139 (5.49)</td>
<td>-136 (-5.36)</td>
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<td>-136 (-5.36)</td>
<td>150 (5.90)</td>
<td>-154 (-6.05)</td>
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<tr>
<td>3rd</td>
<td>N/A</td>
<td>N/A</td>
<td>139 (5.48)</td>
<td>-136 (-5.36)</td>
<td>139 (5.48)</td>
<td>-136 (-5.37)</td>
<td>151 (5.96)</td>
<td>-154 (-6.05)</td>
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<tr>
<td>1st maximum curvature, rad/10^6 mm (rad/10^6 in.)</td>
<td>N/A</td>
<td>N/A</td>
<td>76.1 (1932)</td>
<td>-78.0 (-1982)</td>
<td>91.7 (2330)</td>
<td>-77.4 (-2330)</td>
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<td>N/A</td>
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<tr>
<td>3rd</td>
<td>N/A</td>
<td>N/A</td>
<td>75.3 (1913)</td>
<td>-80.0 (-2120)</td>
<td>92.0 (2336)</td>
<td>-83.1 (-2110)</td>
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<td>5</td>
<td>6</td>
<td>3</td>
<td>5</td>
<td>3</td>
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<tr>
<td>applied lateral load, kN (kips)</td>
<td>229 (51.5)</td>
<td>-197 (-44.4)</td>
<td>238 (53.4)</td>
<td>-215 (-48.4)</td>
<td>217 (48.7)</td>
<td>-220 (-49.5)</td>
<td>228 (51.3)</td>
<td>-201 (-45.2)</td>
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<tr>
<td>lateral displacement, mm (in.)</td>
<td>51 (1.99)</td>
<td>-40 (-1.58)</td>
<td>172 (6.78)</td>
<td>-113 (-4.44)</td>
<td>139 (5.48)</td>
<td>-67 (-2.63)</td>
<td>125 (4.93)</td>
<td>-76 (-3.01)</td>
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<tr>
<td>maximum curvature, rad/10^6 mm (rad/10^6 in.)</td>
<td>28.6 (758)</td>
<td>-25.2 (-639)</td>
<td>87.3 (2218)</td>
<td>-62.3 (-1583)</td>
<td>91.7 (2330)</td>
<td>-27.9 (-708)</td>
<td>N/A</td>
<td>-45.3 (-1150)</td>
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</table>
Table 5.1 (d) Observed lateral load, displacement, and curvature of axial flexural test specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>F0</th>
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<th>F2</th>
<th>F3</th>
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<tr>
<td>Retrofit details mm (in.)</td>
<td>non-retrofit control specimen</td>
<td>3 plies @ 500-1000 (20-39)</td>
<td>2 plies @ 500-1000 (20-39)</td>
<td>1 ply @ 500-1000 (20-39)</td>
</tr>
<tr>
<td>Height of Applied Lateral Load, h, mm (in.)</td>
<td>2369 (93.25)</td>
<td>2419 (95.25)</td>
<td>2419 (95.25)</td>
<td>2413 (95.00)</td>
</tr>
<tr>
<td>Axial load, P, kN (kips)</td>
<td>0.25A_g f_c' = 1299 (292)</td>
<td>0.26A_g f_c' = 1339 (301)</td>
<td>0.26A_g f_c' = 1352 (304)</td>
<td>0.26A_g f_c' = 1334 (300)</td>
</tr>
<tr>
<td>Applied lateral displacement, mm (in.)</td>
<td>229 (51.5)</td>
<td>-193 (-43.4)</td>
<td>237 (53.2)</td>
<td>-212 (-47.7)</td>
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<tr>
<td>Maximum Sustainable Lateral Load, 80% Max. Load</td>
<td>200 (45.0)</td>
<td>-155 (-34.9)</td>
<td>226 (50.8)</td>
<td>-200 (-44.9)</td>
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<tr>
<td>Maximum curvature, rad/10^6 mm (rad/10^6 in.)</td>
<td>51 (1.99)</td>
<td>-51 (-1.99)</td>
<td>189 (743)</td>
<td>-173 (-6.80)</td>
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<td>maximum curvature, rad/10^6 mm (rad/10^6 in.)</td>
<td>51 (1.99)</td>
<td>-52 (-2.06)</td>
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<td>-185 (-7.27)</td>
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<td>Maximum curvature, rad/10^6 mm (rad/10^6 in.)</td>
<td>28.6 (728)</td>
<td>-25.2 (-639)</td>
<td>92.9 (2360)</td>
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<td>Maximum curvature, rad/10^6 mm (rad/10^6 in.)</td>
<td>31.0 (787)</td>
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<td>-153.3 (-3894)</td>
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<td>N/O</td>
<td>N/O</td>
<td>2nd cycle 90_y</td>
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<td>Jacket Rupture</td>
<td>N/A</td>
<td>N/A</td>
<td>N/O</td>
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N/O = not observed
N/A = not applicable
Table 5.2 (a) Observed lateral load, displacement, and curvature of ductile shear test specimens.

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<th>Specimen</th>
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<tr>
<td>Retrofit details mm (in.)</td>
<td>non-retrofit control specimen</td>
<td>2 plies 500 (20)</td>
<td>4 plies 500 (20)</td>
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<tr>
<td>Height of Applied Lateral Load, h, mm (in.)</td>
<td>1524 (60.00)</td>
<td>1524 (60.00)</td>
<td>1537 (60.50)</td>
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<tr>
<td>Axial load, P, kN (kips)</td>
<td>$0.25A\sqrt{f'_c} = 1308 (294)$</td>
<td>$0.26A\sqrt{f'_c} = 1343 (302)$</td>
<td>$0.22A\sqrt{f'_c} = 1334 (300)$</td>
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<th>Half Cycle</th>
<th>N</th>
<th>S</th>
<th>N</th>
<th>S</th>
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<td>applied lateral load, kN (kips)</td>
<td>277 (62.3)</td>
<td>-258 (-58.1)</td>
<td>305 (68.5)</td>
<td>-278 (-62.6)</td>
<td>292 (65.7)</td>
<td>-286 (-64.2)</td>
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<td>3rd</td>
<td>263 (59.1)</td>
<td>-239 (-53.7)</td>
<td>292 (65.7)</td>
<td>-270 (-60.7)</td>
<td>283 (63.6)</td>
<td>-272 (-61.1)</td>
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<td>lateral displacement, $\delta$, mm (in.)</td>
<td>12 (0.49)</td>
<td>-12 (-0.49)</td>
<td>13 (0.50)</td>
<td>-13 (-0.52)</td>
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<td>-13 (-0.50)</td>
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<td>3rd</td>
<td>13 (0.50)</td>
<td>-12 (-0.49)</td>
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<td>-13 (-0.52)</td>
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<td>-13 (-0.50)</td>
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<td>Yield, $\delta$, maximum curvature, rad/10$^6$ mm (rad/10$^6$ in.)</td>
<td>13.9 (354)</td>
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<td>8.2 (211)</td>
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<td>3rd</td>
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<td>-12.5 (-318)</td>
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<td>-9.1 (-231)</td>
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Table 5.2 (b) Observed lateral load, displacement, and curvature of ductile shear test specimens.

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<tr>
<td>Retrofit details mm (in.)</td>
<td>non-retrofit control specimen</td>
<td>2 plies 500 (20)</td>
<td>4 plies 500 (20)</td>
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<tr>
<td>Height of Applied Lateral Load, h, mm (in.)</td>
<td>1524 (60.00)</td>
<td>1524 (60.00)</td>
<td>1537 (60.50)</td>
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<tr>
<td>Axial load, P, kN (kips)</td>
<td>$0.25A_g f'_c = 1308 (294)$</td>
<td>$0.26A_g f'_c = 1343 (302)$</td>
<td>$0.22A_g f'_c = 1334 (300)$</td>
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<td>Half Cycle</td>
<td>N</td>
<td>S</td>
<td>N</td>
</tr>
<tr>
<td>1st applied lateral load, kN (kips)</td>
<td>342 (77.0)</td>
<td>-303 (-68.1)</td>
<td>338 (76.0)</td>
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<td>285 (64.1)</td>
<td>-268 (-60.3)</td>
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<td>1st lateral displacement, $\delta_2$, mm (in.)</td>
<td>25 (0.98)</td>
<td>-26 (-1.01)</td>
<td>25 (1.00)</td>
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<td>3rd</td>
<td>26 (1.02)</td>
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<td>25 (1.00)</td>
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<td>1st maximum curvature, rad/10^6 mm (rad/10^6 in.)</td>
<td>30.2 (767)</td>
<td>-19.9 (506)</td>
<td>13.1 (332)</td>
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<tr>
<td>3rd</td>
<td>34.6 (880)</td>
<td>-23.6 (-600)</td>
<td>22.0 (559)</td>
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Table 5.2 (c) Observed lateral load, displacement, and curvature of ductile shear test specimens.

<table>
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<td>4 plies 500 (20)</td>
</tr>
<tr>
<td>Height of Applied Lateral Load, h, mm (in.)</td>
<td>1524 (60.00)</td>
<td>1524 (60.00)</td>
<td>1537 (60.50)</td>
</tr>
<tr>
<td>Axial load, P, kN (kips)</td>
<td>0.25(A_g f'_c) = 1308 (294)</td>
<td>0.26(A_g f'_c) = 1343 (302)</td>
<td>0.22(A_g f'_c) = 1334 (300)</td>
</tr>
<tr>
<td>Height of Applied Lateral Load, (h), mm (in.)</td>
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<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Axial load, P, kN (kips)</td>
<td>0.25(A_g f'_c) = 1308 (294)</td>
<td>0.26(A_g f'_c) = 1343 (302)</td>
<td>0.22(A_g f'_c) = 1334 (300)</td>
</tr>
<tr>
<td>Specimen</td>
<td>DV0</td>
<td>DV1</td>
<td>DV2</td>
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<tr>
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<td>-----</td>
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<tr>
<td>Height of Applied Lateral Load, h, mm (in.)</td>
<td>1524 (60.00)</td>
<td>1524 (60.00)</td>
<td>1537 (60.50)</td>
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<tr>
<td>Axial load, P, kN (kips)</td>
<td>0.25(A_g f'_c) = 1308 (294)</td>
<td>0.26(A_g f'_c) = 1343 (302)</td>
<td>0.22(A_g f'_c) = 1334 (300)</td>
</tr>
<tr>
<td>Height of Applied Lateral Load, (h), mm (in.)</td>
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<td>0</td>
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<tr>
<td>Axial load, P, kN (kips)</td>
<td>0.25(A_g f'_c) = 1308 (294)</td>
<td>0.26(A_g f'_c) = 1343 (302)</td>
<td>0.22(A_g f'_c) = 1334 (300)</td>
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<td>Maximum Lateral Load</td>
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<td>Displ. ductility</td>
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<td>2</td>
<td>7</td>
</tr>
<tr>
<td>Applied lateral load, kN (kips)</td>
<td>342 (77.0)</td>
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<td>-365 (82.1)</td>
</tr>
<tr>
<td>Lateral displacement, mm (in.)</td>
<td>25 (0.98)</td>
<td>-26 (-1.01)</td>
<td>-92 (3.62)</td>
</tr>
<tr>
<td>Maximum curvature, rad/10^4 mm (rad/10^6 in.)</td>
<td>30.2 (767)</td>
<td>-19.9 (-506)</td>
<td>N/A</td>
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</table>
Table 5.2 (d) Observed lateral load, displacement, and curvature of ductile shear test specimens.

<table>
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<th>DV1</th>
<th>DV2</th>
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<tr>
<td>Retrofit details</td>
<td>non-retrofit control specimen</td>
<td>2 plies 500 (20)</td>
<td>4 plies 500 (20)</td>
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<td>Height of Applied Lateral Load, h, mm (in.)</td>
<td>1524 (60.00)</td>
<td>1524 (60.00)</td>
<td>1537 (60.50)</td>
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<tr>
<td>Axial load, P, kN (kips)</td>
<td>$0.25A_{g}f'_{c} = 1308 (294)$</td>
<td>$0.26A_{g}f'_{c} = 1343 (302)$</td>
<td>$0.22A_{g}f'_{c} = 1334 (300)$</td>
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<tr>
<td>Half Cycle</td>
<td>N</td>
<td>S</td>
<td>N</td>
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<tr>
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<td>4</td>
<td>7</td>
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<tr>
<td>applied lateral load, kN (kips)</td>
<td>1&lt;sup&gt;st&lt;/sup&gt;</td>
<td>342 (77.0)</td>
<td>-303 (-68.1)</td>
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<tr>
<td></td>
<td>3&lt;sup&gt;rd&lt;/sup&gt;</td>
<td>285 (64.1)</td>
<td>-268 (-60.3)</td>
</tr>
<tr>
<td>lateral displacement, mm (in.)</td>
<td>1&lt;sup&gt;st&lt;/sup&gt;</td>
<td>25 (0.98)</td>
<td>-26 (-1.01)</td>
</tr>
<tr>
<td></td>
<td>3&lt;sup&gt;rd&lt;/sup&gt;</td>
<td>26 (1.02)</td>
<td>-25 (-0.99)</td>
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<tr>
<td>maximum curvature, rad/10&lt;sup&gt;6&lt;/sup&gt; mm (rad/10&lt;sup&gt;6&lt;/sup&gt; in.)</td>
<td>1&lt;sup&gt;st&lt;/sup&gt;</td>
<td>30.2 (767)</td>
<td>-19.9 (-506)</td>
</tr>
<tr>
<td></td>
<td>3&lt;sup&gt;rd&lt;/sup&gt;</td>
<td>34.6 (880)</td>
<td>-23.6 (-600)</td>
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<td>N/O</td>
<td>N/O</td>
<td>3&lt;sup&gt;rd&lt;/sup&gt; cycle at $8\delta_y$</td>
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<td>Jacket Rupture</td>
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<td>N/A</td>
<td>N/O</td>
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Table 5.3 (a) Observed transverse strains in retrofit jackets of axial flexural test specimens, microstrain.

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<th>Specimen</th>
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<td>3 plies 500-1000 (20-39)</td>
<td>2 plies 500-1000 (20-39)</td>
<td>1 ply 500-1000 (20-39)</td>
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<tr>
<td>2419 (95.25)</td>
<td>2419 (95.25)</td>
<td>2413 (95.00)</td>
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<td>Axial load (kips)</td>
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<td>0.26Afg' = 1352 (304)</td>
<td>0.26Afg' = 1334 (300)</td>
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<tr>
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<td>307</td>
<td>71</td>
<td>426</td>
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<tr>
<td>3rd</td>
<td>331</td>
<td>307</td>
<td>449</td>
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<td></td>
<td></td>
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<tr>
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<td>284</td>
<td>95</td>
<td>213</td>
</tr>
<tr>
<td>3rd</td>
<td>118</td>
<td>95</td>
<td>284</td>
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<tr>
<td>Maximum strain on tension face</td>
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<td></td>
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<tr>
<td>1st</td>
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<td>355</td>
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<td>Yield, δy</td>
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Table 5.3 (b) Observed transverse strains in retrofit jackets of axial flexural test specimens, microstrain.

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<th>F3</th>
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<td>3 plies 500-1000 (20-39)</td>
<td>2 plies 500-1000 (20-39)</td>
<td>1 ply 500-1000 (20-39)</td>
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<td>2419 (95.25)</td>
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<td>0.26A_gf_c' = 1352 (304)</td>
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<th>N</th>
<th>S</th>
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<td>851</td>
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<td>851</td>
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<td>402</td>
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<td>780</td>
<td>875</td>
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<td>615</td>
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<td>0</td>
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<td>142</td>
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Table 5.3 (c) Observed transverse strains in retrofit jackets of axial flexural test specimens, microstrain.

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<tr>
<td>3 plies 500-1000 (20-39)</td>
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<td>2419 (95.25)</td>
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Table 5.3 (d) Observed transverse strains in retrofit jackets of axial flexural test specimens, microstrain.

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<td>2 plies 500 (20)</td>
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<tr>
<td>mm (in.)</td>
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<td>2 plies 500-1000 (20-39)</td>
<td>1 ply 500-1000 (20-39)</td>
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<td>Height of Applied Lateral Load, mm (in.)</td>
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<td>2419 (95.25)</td>
<td>2413 (95.00)</td>
</tr>
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<td>Axial load (kips)</td>
<td>$0.26A_{g}f'_{c} = 1339 (301)$</td>
<td>$0.26A_{g}f'_{c} = 1352 (304)$</td>
<td>$0.26A_{g}f'_{c} = 1334 (300)$</td>
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<th>N</th>
<th>S</th>
<th>N</th>
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<td>2648</td>
<td>4422</td>
<td>4611</td>
<td>5817</td>
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<td>2294</td>
<td>4422</td>
<td>5959</td>
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Table 5.4 (a) Observed transverse strains in retrofit jacket of ductile shear test specimens, microstrain.

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<td>2 plies 500-1000 (20-39)</td>
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<tr>
<td>Height of Applied Lateral Load, mm (in.)</td>
<td>1524 (60.00)</td>
<td>1537 (60.50)</td>
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<td>Axial load (kips)</td>
<td>0.26A_{f_e}' = 1343 (302)</td>
<td>0.22A_{f_e}' = 1334 (300)</td>
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<td>Half Cycle</td>
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<td>426</td>
<td>568</td>
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<td>402</td>
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<tr>
<td>3rd</td>
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Table 5.4 (b) Observed transverse strains in retrofit jacket of ductile shear test specimens, microstrain.

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<td>Axial load (kips)</td>
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<th>1st</th>
<th>3rd</th>
<th>1st</th>
<th>3rd</th>
<th>1st</th>
<th>3rd</th>
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<td>47</td>
<td>95</td>
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N | S | N | S
---|---|---|---
709 | 946 | 3239 | 591
615 | 946 | 3169 | 615
591 | 331 | 544  | 544
638 | 355 | 591  | 568
473 | 355 | 402  | 402
497 | 355 | 426  | 402
166 | 237 | 0    | 95
237 | 307 | 47   | 95
Table 5.4 (c) Observed transverse strains in retrofit jacket of ductile shear test specimens, microstrain.

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<td>(in.)</td>
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<td>0.26( f_c' ) = 1343 (302)</td>
<td>0.22( f_c' ) = 1334 (300)</td>
</tr>
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<td>N: maximum strain on compression face</td>
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<td>S: maximum strain on tension face</td>
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<td>1773</td>
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<tr>
<td></td>
<td>S: maximum strain on east face</td>
<td>S: strain at center of east face</td>
</tr>
<tr>
<td></td>
<td>1st: 1490</td>
<td>1632</td>
</tr>
<tr>
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<td>2nd: 1655</td>
<td>1608</td>
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<tr>
<td></td>
<td>Maximum displacement ductility</td>
<td>Maximum strain on east face</td>
</tr>
<tr>
<td></td>
<td>7</td>
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<td>1632</td>
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<tr>
<td></td>
<td>Maximum strain on compression face</td>
<td>Maximum strain on tension face</td>
</tr>
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<td>Maximum strain on east face</td>
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Table 5.4 (d) Observed transverse strains in retrofit jacket of ductile shear test specimens, microstrain.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>DV1</th>
<th>DV2</th>
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<tbody>
<tr>
<td>Retrofit details mm (in.)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 plies 500 (20)</td>
<td>4 plies 500 (20)</td>
<td></td>
</tr>
<tr>
<td>1 plies 500-1000 (20-39)</td>
<td>2 plies 500-1000 (20-39)</td>
<td></td>
</tr>
<tr>
<td>Height of Applied Lateral Load, mm (in.)</td>
<td>Excursion</td>
<td></td>
</tr>
<tr>
<td>1524 (60.00)</td>
<td>1537 (60.50)</td>
<td></td>
</tr>
<tr>
<td>Axial load (kips)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.26A_{g}f'_{c} = 1343 (302)</td>
<td>0.22A_{g}f'_{c} = 1334 (300)</td>
<td></td>
</tr>
<tr>
<td>Half Cycle</td>
<td>N</td>
<td>S</td>
</tr>
<tr>
<td>displacement ductility</td>
<td>7</td>
<td>8</td>
</tr>
<tr>
<td>Maximum Sustainable Lateral Load, 80% Max. Load</td>
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<td></td>
</tr>
<tr>
<td>maximum strain on compression face</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1st</td>
<td>5699</td>
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</tr>
<tr>
<td>3rd</td>
<td>5699</td>
<td>3972</td>
</tr>
<tr>
<td>maximum strain on tension face</td>
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<td>1st</td>
<td>3854</td>
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<td>3rd</td>
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<td>maximum strain on east face</td>
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<td>1821</td>
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<tr>
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<td>1986</td>
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Figure 5.1 Orientation of column and lateral load.
Figure 5.2 Applied load vs. displacement.

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Figure 5.2 (continued) Applied load vs. displacement.
Figure 5.2 (continued) Applied load vs. displacement.
Figure 5.3 Jacket bulging, buckling, and rippling.

(a) Bulging
- Bulging more continuous along height
- No discontinuity in jacket

(b) Buckling
- Buckling more localized
- Discontinuity in jacket

(c) Rippling
- Rippling
Figure 5.4 Specimen F0 at $2.5\delta_y$. 
Figure 5.4 Specimen F0 at 2.58\,µ.
Figure 5.5 Specimen F1 at 88°.
Figure 5.5 Specimen F1 at 85\degree.
Figure 5.6 Specimen F2 at 85.
Figure 5.6 Specimen F2 at 86.

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Figure 5.7 Specimen F3 before jacket rupture.

Figure 5.8 Specimen F3 after jacket rupture
Figure 5.7  Specimen F3 before jacket rupture.

Figure 5.8  Specimen F3 after jacket rupture
Figure 5.9 Specimen DV0 crushed concrete and shear crack.

Figure 5.10 Specimen DV0 longitudinal reinforcing bar buckling.
Figure 5.9 Specimen DV0 crushed concrete and shear crack.

Figure 5.10 Specimen DV0 longitudinal reinforcing bar buckling.
Figure 5.10 Specimen DV0 longitudinal reinforcing bar buckling.
Figure 5.11 Specimen DVI after jacket rupture.

Figure 5.12 Specimen DVI at 68°.
Figure 5.11 Specimen DV1 after jacket rupture.

Figure 5.12 Specimen DV1 at 63.
Figure 5.13  Specimen DV2 after jacket rupture.
Figure 5.13 Specimen DV2 after jacket rupture.
Figure 5.14 Shear cracks in Specimens DV0 and DV1.
Figure 5.14 Shear cracks in Specimens DV0 and DV1.
CHAPTER 6

DISCUSSION OF TEST RESULTS

This chapter discusses the results of tests presented in the previous chapter. The test results for the non-retrofit and retrofit specimens are compared to quantify the improvement in behavior provided by each jacket retrofit. The test results for the retrofit specimens are compared to evaluate the influence of the number of plies of carbon tow sheet in the jackets. The tests results are also compared with the analytical results for the retrofit and non-retrofit specimens presented in Chapter 4. The chapter is organized into four main sections. Section 6.1 compares the overall behavior of the retrofit and non-retrofit test specimens, considering stiffness, strength, and ductility. Section 6.2 discusses local behavior of the specimens. Section 6.3 compares analytical results presented in Chapter 4 with the test results. Section 6.4 summarizes the discussion of the test results.

6.1 OVERALL BEHAVIOR

This section discusses the overall experimental behavior of the retrofit and non-retrofit specimens. The jackets used to retrofit the test specimens were designed to increase the flexural ductility of the column specimens without increasing the flexural strength or stiffness. This type of retrofit is useful in cases where increasing the flexural strength or stiffness of the columns would increase the forces that develop in other components of a non-ductile reinforced concrete building structure (e.g., beams and beam-column joints) to levels that would require these other components to be strengthened.

The strength, stiffness, ductility, and energy dissipation of the non-retrofit and retrofit specimens are discussed below. The influence of the number of jacket plies on the strength,
stiffness, ductility, and energy dissipation is evaluated. The cracking moment and shear demand are also discussed.

6.1.1 Strength

Figure 5.2 shows the load vs. displacement response curves of both the retrofit and non-retrofit test specimens. Figure 6.1 (a.) shows backbone load vs. displacement response curves for Specimens F0, F1, F2, and F3 (the axial flexural specimens). These backbone response curves were generated by plotting the points of maximum lateral displacement and corresponding applied lateral load for the first cycle at each displacement ductility level. The backbone curve for each axial flexural specimen reaches a plateau at approximately the same lateral load capacity of 228 ± 10 kN (see also Table 5.1). Figure 6.1 (b) shows the backbone load vs. displacement for Specimens DV0, DV1, and DV2. Each ductile shear specimen has approximately the same lateral load capacity of 361 ± 20 kN (see also Table 5.2). The strength is approximately the same for both the retrofit and non-retrofit columns.

6.1.2 Stiffness

The slope of the load vs. displacement response curve defines the stiffness of the column specimens. The initial slope of the backbone load vs. displacement response curves for the retrofit and non-retrofit specimens is the same, as shown in Figure 6.1 (a) and (b). The initial stiffness of each test specimen was calculated from the loading cycles to 0.66M_y. A linear regression of the load and displacement data points between 20% and 80% of the peak load during the 0.66M_y cycles was performed to define the initial stiffness of each specimen. The stiffness calculation was made for the first north and south cycles, and the third north and south cycles. The results from the north cycles are reported in Table 6.1.

The theoretical stiffness of the test specimens is \( K = 3 \cdot E \cdot I_e / L^3 \) where \( I_e \) is an effective
moment of inertia of the column cross section. Several effective moments of inertia were considered as shown in Table 6.1. The effective moment of inertia from the ACI 318-95 code specifications was calculated as:

\[
I_{e,\text{ACI}} = I_{ut} \left( \frac{M_{cr}}{M_a} \right)^3 + I_{ct} \left( 1 - \left( \frac{M_{cr}}{M_a} \right)^3 \right)
\]  

(6.1)

where: 
- \( I_{ut} \) = the uncracked transformed section moment of inertia;
- \( I_{ct} \) = the cracked transformed section moment of inertia;
- \( M_{cr} \) = the moment at which the concrete first cracks; and,
- \( M_a = 0.66M_y \).

Other effective moments of inertia equal to \( I_{ut} \), \( I_{ct} \), and the gross section moment of inertia, \( I_g \), were also considered. To accurately estimate the cracked transformed section moment of inertia, a fiber section analysis was performed using the program RESPONSE (Collins et al. 1991) at a moment of 0.66\( M_y \) and the constant axial load level used in the tests (\( P = 300 \) kN (67.4 kips)) to determine the position of the neutral axis. The cracked transformed section moment of inertia corresponding to this neutral axis position, \( I_{ct} \), was calculated and also used as an effective moment of inertia. The results in Table 6.1 show that the theoretical values of stiffness obtained from the section analyses provide bounds on the experimental stiffness values. The results based on \( I_g \) provide an upper bound, and the results based on \( I_c \) from ACI 318-95 provide a lower bound.

The stiffnesses of the non-retrofit and retrofit columns during cycles to \( 2\delta_y \), \( 4\delta_y \), and \( 6\delta_y \) were compared as shown in Figure 6.2. This figure shows that the slope of the load vs. displacement response curves are similar at various displacement ductilities. The results in Figure
6.2 and Table 6.1 show that the non-retrofit and retrofit column specimens have similar stiffnesses. The number of plies in the jacket does not have a significant influence on the stiffness of the columns.

6.1.3 Ductility

The displacement ductility of the retrofit specimens was much greater than the displacement ductility of the non-retrofit specimens. The displacement ductility capacity of each test specimen was given in Table 5.1 for the axial flexural test specimens and in Table 5.2 for the ductile shear test specimens as the displacement ductility at the maximum sustainable lateral load (80% of the maximum load). To show the trend in this data, the displacement ductility capacity is plotted vs. the number of jacket plies in Figure 6.3. The results for the axial flexural specimens are shown in Figure 6.3 (a) and the results for the ductile shear specimens are shown in Figure 6.3 (b).

Figure 6.3 shows the increases in displacement ductility with increases in jacket stiffness and strength as controlled by the number of jacket plies. It should be noted that the ductility of the test specimens was limited by four different failure modes: (1) spalling and crushing of poorly confined concrete, and buckling of the longitudinal reinforcing bars under flexure, observed in Specimen F0; (2) cracking and crushing of the concrete under flexure which degrades the shear capacity and results in a sliding failure along a shear crack, observed in Specimen DVO; (3) rupture of the jacket, observed in Specimen F3; and (4) rupture of the longitudinal reinforcing bars as a result of low cycle fatigue from buckling and straightening of the bars, observed in Specimens F1, F2, DV1, and DV2. Specimen F3 was the only specimen for which the ductility was controlled by jacket strength. Thus, it appears that when the jacket strength is sufficient, stiffness is the jacket property that influences the ductility of the retrofit specimen. The maximum
ductility which can be achieved using a FRP jacket retrofit on a square reinforced concrete column is limited by buckling and subsequent rupture of the longitudinal reinforcing bars, which is not significantly influenced by increasing jacket stiffness.

6.1.4 Energy Dissipation

The energy dissipated by the test specimens was calculated by integrating the lateral load vs. displacement curve. It was assumed that the chord length of the column remained constant, and the axial load did not contribute to the energy dissipation. The energy dissipation was quantified as the energy dissipated per half cycle. The energy dissipated per half cycle, $E_{1/2\:\text{cycle}}$, was normalized by a hypothetical elastic energy, $E_{\text{el}}$, value at yield, calculated as $1/2 \: K \: (\delta_y)^2$ as shown in Figure 6.4. As previously reported in Table 5.1, Specimens F0 and F3 had yield displacements, $\delta_y$, of 25 mm (1.0 in.), whereas Specimens F1 and F2 had yield displacements of 23 mm (0.9 in.). The normalization with respect to $\delta_y$ eliminated this difference. The normalized energy dissipation per half cycle was calculated as follows:

$$E_n = \frac{E_{1/2\:\text{cycle}}}{E_{\text{el}}} = \frac{\sum \frac{1}{2} (H_{i+1} + H_i) (\delta_{i+1} - \delta_i)}{\frac{1}{2} (K \: \delta_y^2)}$$

(6.1)

where: $H$ = the applied lateral load;

$\delta$ = the lateral displacement corresponding to the applied lateral load;

$K$ = the theoretical initial stiffness of the column using $I_c = I_{cd}$; and,

$\delta_y$ = the lateral yield displacement.
The normalized energy dissipation per half cycle is shown in Figure 6.5 (a) for the axial flexural specimens and in Figure 6.5 (b) for the ductile shear specimens. The energy dissipation increases with increased ductility level until failure occurs. Because the failures of the specimens are not preceded by deterioration in the hysteretic behavior in the cycles before failure, a decrease in energy dissipation per half cycle is not observed before failure. Figure 6.5 (a) indicates that Specimen F2 with a four ply jacket behaved as well as Specimen F1 with a six ply jacket through a displacement ductility of \(7\delta_y\).

As previously stated, the jacket does not increase the strength or stiffness of the specimens. The longitudinal reinforcement in the specimens controls the moment capacity and the strength of the specimens. Each test specimen has the same longitudinal reinforcement, and approximately the same strength. Therefore, as long as the hysteretic behavior does not deteriorate, each specimen dissipates the same energy at the same ductility level. Therefore, the energy dissipation per half cycle is independent of the number of plies in the jacket.

6.1.5 Cracking Moment

The non-retrofit specimens exhibited flexure and flexure-shear cracks typical of reinforced concrete building columns subjected to combined axial and reverse cyclic lateral loading. Crack patterns for Specimen F0 and Specimen DV0 are shown in Figure 6.6, Table 6.2 indicates the location of the cracks in the non-retrofit and retrofit specimens. The typical spacing between flexure and flexure-shear cracks for the non-retrofit specimens was 127 mm ± 51 mm. The typical spacing between flexure and flexure shear cracks for the retrofit specimens was 178 mm ± 25 mm. The cracks in the retrofit specimen usually coincided with the locations of the transverse reinforcement (ties) inside the column. The decrease in spacing between cracks and
the increased ductility in the retrofit specimens resulted in large crack widths, which exceeded 4 mm (0.16 in) in some cases. However, initial cracking occurred later in the retrofit specimens.

Initial cracking of the non-retrofit and retrofit specimens occurred at the column footing interface, which is the location of maximum moment. The jackets in the retrofit specimens were provided a 13 mm (0.5 in.) gap to prevent the jacket from bearing on the footing. Therefore, the cracking moment at the column footing interface was not increased by the presence of the jacket in the retrofit specimens. The cracking moment was calculated using the ACI 318-95 code provisions and determined to be 114 kN m (84 k ft). The results in Table 6.2 show that the cracking moment in the tests exceeded the predicted cracking moment from ACI 318-95. As the applied lateral load was increased, the second flexural crack in the retrofit specimens occurred at a level of load larger than the load at which the second flexural crack formed in the non-retrofit specimens. The formation of the second crack within the jacketed region was determined from a jump in the longitudinal strain on the tension face of the column. Table 6.2 indicates that flexural cracking in the jacketed portion of the retrofit specimens occurs at a moment that exceeds the cracking moment of the non-retrofit columns.

6.1.6 Decrease in Shear Demand with Increased Ductility

The set-up used to test the specimens resulted in a decreasing shear demand on the test specimens with increasing lateral displacement after the plastic hinge forms at the column base. The decrease in shear demand results from an increase in the P-Δ effect, but this decrease in shear is not reflected in the lateral load because of the static conditions of the test set-up. The decrease in shear demand with increasing lateral displacement also occurs in building columns which form a plastic hinge and are subjected to significant P-Δ effects. Figure 6.7 shows the loading
After a plastic hinge has formed at the base of the building column, the moment at the top of the column is a function of the location of the inflection point:

\[ M = M_p \left( \frac{1}{\alpha} - 1 \right) \]  

(6.2)

where: \( \alpha = \) the ratio of the height of the inflection point to the total height of the building column;

\( M = \) the moment at the top of the column; and

\( M_p = \) the plastic moment capacity of the column.

Moment equilibrium of the building column requires:

\[ M + M_p = \frac{P \Delta_2}{2} + H \frac{h_2}{2} \]  

(6.3)

where: \( P = \) the axial load;

\( \Delta_2 = \) the lateral displacement at the top of the column;

\( H = \) the applied lateral load; and,

\( h_2 = \) the height of the column.

The shear in the column, \( V \), is equal to the lateral load, \( H \). Substituting \( V \) for \( H \) in Equation 6.3 and substituting Equation 6.2 for \( M \) in Equation 6.3, results in an equation that can be solved for \( V \):

\[ V = \frac{M_p}{\alpha h_2} - \frac{P \Delta_2}{h_2^2} \]  

(6.4)
After a plastic hinge has formed at the base of the test specimen, the moment at the base of the test specimen is:

\[ M_p = h_1 H = \beta h_3 H \]  \hspace{1cm} (6.5)

where:  \( h_1 \) = the height of the lateral load;
\( h_3 \) = the height of the application of axial load for the test specimen; and,
\( \beta \) = the ratio of \( h_1 \) to \( h_3 \).

Note that the axial load, \( P \), has no moment arm about the base of the test specimen.

However, the lateral displacement \( \Delta_3 \) produces a horizontal component of \( P \), which, assuming \( \tan (\Delta_3 / h_3) \) is approximately equal to \( \Delta_3 / h_3 \), is \( (P \Delta_3) / h_3 \), where \( \Delta_3 \) = the displacement at the point of application of axial load for the test specimens. The shear in the column is:

\[ V = H - \frac{P \Delta_3}{h_3} \]  \hspace{1cm} (6.6)

Solving Equation 6.5 for \( H \) and substituting into Equation 6.6 gives:

\[ V = \frac{M_p}{\beta h_3} - \frac{P \Delta_3}{h_3} \]  \hspace{1cm} (6.7)

Thus the shear in the test specimen is equal to the shear in the building column (Equation 6.4) when the lateral load height, \( h_1 \) equals the height of the inflection point in the building column:

\[ h_1 = \beta h_3 = \alpha h_2 \]  \hspace{1cm} (6.7)
and when:

\[
\frac{\Delta_2}{h_2} = \frac{\Delta_3}{h_3}
\]  

(6.8)

Equation 6.7 represents the basic assumption relating the test specimens to building columns. To evaluate the relationship in Equation 6.8, the ratio of displacement to height was plotted for Specimen DV2. During the test of this specimen, the displacement at the point of applied lateral load, \(\Delta_1\), and the displacement at the point of applied axial load, \(\Delta_3\), were measured. The ratio of the displacement to the corresponding height was calculated at each displacement ductility level and is shown in Figure 6.8. The values of \(\Delta_1/h_1\) and \(\Delta_3/h_3\) are approximately equal as indicated in Figure 6.8. From Equation 6.7:

\[
\frac{\Delta_1}{h_1} = \frac{\Delta_1}{\alpha h_2}
\]  

(6.9)

If it is assumed that \(\Delta_1 = \alpha \Delta_2\), then:

\[
\frac{\Delta_1}{h_1} = \frac{\alpha \Delta_2}{\alpha h_2} = \frac{\Delta_2}{h_2}
\]  

(6.10)

thus:

\[
\frac{\Delta_2}{h_2} = \frac{\Delta_3}{h_3}
\]  

(6.11)
The assumption $\Delta_1 = \alpha \Delta_2$ is valid when $\alpha = 0.5$ (the point of inflection is at the half height of the building column) and when $\alpha = 1.0$ (the point of inflection is at the top of the building column). In other cases, an error is introduced by elastic deformations in the column, but this error should be small. Therefore, the shear demand in the test specimens is similar to the shear demand in building columns.

6.2 LOCAL BEHAVIOR

This section discusses load behavior of non-retrofit and retrofit specimens. Flexural deformations (curvature) along the height of the specimens and strains in the jackets of the specimens are the primary focus. In the following section these results are compared to analytical results discussed in Chapter 4.

6.2.1 Curvature

The curvatures over the height of the test specimens are presented in Figure 6.9. The curvatures were obtained from linear potentiometers on the west face or on the north and south faces of the test specimen. Values of curvature at specific points in the load history of the specimens were presented in Table 5.1 for the axial flexural specimens and Table 5.2 for the ductile shear specimens. The curvatures plotted in Figure 6.9 were calculated at the maximum displacement in the first north cycle and the first south cycle at each displacement ductility level until failure for the non-retrofit specimens and up to $6\delta_y$ for the retrofit specimens.

As expected, the curvature is the largest in the plastic hinge region at the column base. The curvature decreases with distance from the column footing interface. At failure of the non-retrofit specimens, the curvature is quite small at the base. The retrofit specimens were able to reach large values of curvature as indicated in Table 5.1 for the axial flexural specimens and Table 5.2 for the ductile shear specimens, and as shown in Figure 6.9. For the retrofit specimens,
the largest values of curvature usually occur at the locations of transverse cracks in the jacket. There were two primary locations with large values of curvature: (1) the base of the column, and (2) approximately 178 mm from the column footing interface.

The location of the potentiometers is shown in Figure 4.17 (b) and schematically in Figure 6.9. The values of curvature presented in Tables 5.1 and 5.2, and in Figure 6.9 are average curvatures determined over the gage length of the potentiometers. The curvature values determined from the potentiometers on the west face are calculated for regions centered at 51 mm (2 in.), at 254 mm (10 in.), and at 559 mm (22 in.). The curvature values determined from the potentiometers on the north and south faces are calculated for regions centered at 51 mm (2 in.), at 178 mm (7 in.), at 330 mm (13 in.), at 508 mm (20 in.), and at 711 mm (28 in.). The curvature values calculated for the regions centered at 51 mm (2 in.) are often much larger than the curvature values for other regions along the height of the column. This increased curvature value is attributed to pull out of the longitudinal reinforcing bars from the footing at the base of the column, which is included in the calculated curvature. Therefore, the curvature values at 51 mm (2 in.) do not accurately represent the average curvature of the cross section over the 102 mm (4 in.) gage length. The curvature values for the regions centered at 254 mm (10 in.) or at 178 mm (7 in.) are considered to be an accurate measure of the curvature developed in the non-retrofit and retrofit specimens.

For the non-retrofit specimens, pins to attach the potentiometers were embedded into the core concrete, so that crushing and/or spalling of the cover concrete did not prevent potentiometer data from being obtained. For the retrofit specimens, the potentiometer pins could not be embedded into the core without damaging the jacket. Therefore, the potentiometer pins were attached to the jackets of the retrofit specimens with epoxy. At large displacement ductility
levels, the pins attached to the jacket began to fall off as a result of jacket distortion. Furthermore, bulging and buckling of the jacket rotated the potentiometer pins. Therefore, the ultimate curvature could not be determined for all of the test specimens. The potentiometers on the west face of the test specimens generally provided better data at large displacement ductility levels than the potentiometers on the north and south faces. This data is reported in Table 5.1 and 5.2. The test of Specimen DV1 was an exception in which the potentiometers on the north and south faces provided better data and this data is reported in Table 5.2.

The curvature data for the non-retrofit and retrofit specimens show that the retrofit specimens were significantly more ductile than the non-retrofit specimens. The curvature data and test observations indicate that the retrofit and non-retrofit columns behave similarly until transverse cracks begin to form. At this time, the curvature begins to concentrate at the column footing interface and the first transverse crack in the jacket located at approximately 178 mm.

6.2.2 Jacket Strains

The transverse strains measured by strain gages on the jacket of the retrofit specimens were plotted as continuous curves of strain around the jacket. The plots are generated for the strain gage locations shown in Figure 6.10. The jacket strains are plotted for each displacement ductility level in Figure 6.11. By showing the jacket strains as a continuous curve around the specimen, the strains in the south, east, north, and west faces of the jacket can be compared. The jacket strains were measured at 229 mm or D/2 from the column footing interface. Strains recorded at the maximum displacement in the first and third cycles at each displacement ductility level are shown. The dotted line represents the strains during the first cycle and the solid line represents strains during the third cycle. Strains from the north cycles are plotted on the bottom of the graph and strains from the south cycles are potted on the top. The first cycle shown is 1.
Subsequent cycles to $2\delta_y$, $3\delta_y$, and so on up to $8\delta_y$ are shown. If strain data was obtained from cycles past $8\delta_y$, the displacement ductility increment increased to $2\delta_y$. Figure 6.11 presents the jacket strains for all of the retrofit specimens. As expected, jacket strains increase with increases in displacement ductility. The increase in jacket strain is a result of increasing (and permanent) dilation of the concrete that is occurring beneath the jacket. At ductility levels beyond approximately $3\delta_y$, the jacket strains that develop on a compression face do not completely recover after the compression stress is removed (under cyclic loading), because the concrete dilation strains beneath the jacket do not recover. When the corresponding concrete dilation is observed visually, it is termed “bulging” of the jacket as described in Chapter 5. Evidence of significant jacket bulging in the jacket strain data is noted in Figure 6.11.

Figure 6.12 explains the behavior of the jacket at large levels of displacement ductility. On the tension face, shown in Figure 6.12 (a), transverse cracks open. When the lateral load is reversed and the same face is put into compression and the cracks close. However, the damaged concrete in the vicinity of the column does not return to the original configuration and this results in bulging and buckling as shown in Figure 6.12 (b and c). As the lateral load cycles continue, the damage to the concrete underneath the jacket continues. Figure 6.12 shows the typical location of damaged concrete. Eventually, the longitudinal reinforcing bar becomes unsupported and it begins buckle and straighten under the reversed cyclic lateral loading.

The jacket strain data provides evidence of this behavior. The third cycle at a displacement ductility level usually results in jacket strains that are larger than the strains during the first cycle. The increase in jacket strain during cycles at a particular displacement ductility is a result of continued dilation of the concrete under compression stress underneath the jacket. When the loading is reversed and the compression stress is relieved, the concrete dilation is not
recovered. On the following cycle, further dilation of the concrete takes place. However, as shown in Figure 6.11, the increment in jacket strain between the first cycle (dotted line) and third cycle (solid line) is usually small.

In summary, the jacket strain increases with displacement ductility level and with cyclic loading at a single ductility level. Eventually, the jacket begins to bulge, and this bulging was observed visually and also appears as an increase in jacket strain where the bulging occurs.

6.3 COMPARISON OF EXPERIMENTAL VS. ANALYTICAL BEHAVIOR

This section compares the experimental behavior of non-retrofit and retrofit specimens with the behavior predicted by the analyses described in Chapter 4. The comparison of flexural behavior is given in Section 6.3.1. The comparison of shear behavior is given in Section 6.3.2.

6.3.1 Flexural Behavior

Chapter 4 included analyses of the flexural strength and ductility of the test specimens. This section compares the results of these analyses with the test results. Certain test results related to flexural behavior have already been discussed and compared to analytical results, including the flexural stiffness and the cracking moment. This section concentrates on flexural strength and ductility, and on test results related to flexural ductility, including concrete strains and jacket strains.

**Flexural Response**

The flexural response of the non-retrofit specimen (Specimen F0) was determined analytically in Section 4.1.3. The predicted moment capacity was \( M_n = 396 \text{ kN m} \) (292 k ft). The experimentally obtained moment capacity at the column footing interface was 543 kN m (401 k ft) in a north cycle and 467 kN m (344 k ft) in a south cycle. The average flexural overstrength is 28%. The flexural overstrength can be attributed to several factors. The yield stress of the
longitudinal reinforcement was 460 MPa (66.7 ksi), which is 11% larger than the minimum specified yield stress of 414 MPa (60.0 ksi). In addition, flexural failure of Specimen F0 occurred at a height approximately D/2 (229 mm (9 in.)) from the column footing interface, where the moment demand was only 90% of the moment at the column footing interface. In addition, it is likely that some strain hardening of the longitudinal reinforcement occurred.

As discussed earlier, the flexural strength of the retrofit specimens (Specimens F1, F2 and F3) was similar to that of Specimen F0. Therefore, these test specimens had similar overstrength compared to the analytical prediction.

Experimental vs. Analytical Yield Curvature, $\phi_y$

From the curvature data, a yield curvature was defined as the curvature when the lateral displacement was $\delta_y$. As discussed previously, this displacement was determined from the yielding of the longitudinal reinforcing bars and a flattening of the lateral load vs. displacement response of the test specimen. Yielding of the reinforcing bars was obtained from strain gage data, with the strain gages located as shown in Figure 4.17. Strain gages located at 305 mm (12 in.) above the column footing interface recorded strains above the yield strain at $\delta_y$. Therefore, the yield curvature is defined as the curvature at $\mu = 1$. Values of yield curvature for each test specimen are presented in Table 6.3.

The analytical yield curvature of the test specimens was determined to be $17 \times 10^6$ $1$/mm (Section 4.5.1). Table 6.3 indicates that the experimental values of yield curvature are lower than the analytical yield curvature. The lower experimental yield curvature values can be attributed to: (1) the moment gradient of the column produces a maximum curvature at the column footing interface and the region where the experimental yield curvatures are determined is centered at 254 mm (10 in.) above the interface; (2) the fiber cross-section analysis used to calculate the yield
curvature neglects the tensile strength of the concrete; and, (3) the curvature values are
determined from potentiometers with gage lengths of 305 mm (12 in.), and over part of this length
the uncracked concrete stiffens the section.

Experimental vs. Analytical Curvature, $\phi$, and Curvature Ductility, $\mu_\phi$

The curvature at the maximum lateral displacement in the first north and south cycles
plotted versus the displacement ductility level in Figure 6.13. Curvature values at $2\delta_y$, $4\delta_y$, and
$6\delta_y$ are also given in Table 6.4. In Section 4.5.1, the axial flexural jacket retrofits were designed
for a curvature of $73 \times 10^{-6}$ /mm, corresponding to a required displacement ductility of $\mu_\phi = 2.3$.
The point $\phi = 73 \times 10^{-6}$ /mm, $\mu_\phi = 2.3$ is marked with an “X” on Figure 6.13. The
experimentally determined curvature values have some scatter at each displacement ductility
level. However, Figure 6.13 shows that the analytically predicted curvature at $\mu_\phi = 2.3$ is much
larger than determined experimentally. The lower experimental value of curvature is attributed
to: (1) pull out of the longitudinal reinforcing bars from the footing at the base of the column,
which adds to the displacement, but is not included in the curvature, which is determined for a
region centered at 254 mm (10 in.) above the footing; and (2) the curvature values are average
values for 305 mm (12 in.) gage length centered at 254 mm (10 in.) above the column footing
interface and this average curvature is less than the largest curvatures at the base of the column.
The vertical axis of Figure 6.13 shows values of ultimate curvature calculated in Section 4.5.1
(Table 4.8, $f_{\text{residual}} = 5$ MPa, $e_{\text{c,et}} = e_{\text{cu}}$) for test specimens retrofit with 2 ply jackets (Specimen F3),
4 ply jackets (Specimen F2), and 6 ply jackets (Specimen F1). The figure shows that the
displacement ductility of Specimen F3 ($5\delta_y$) is reached shortly after the analytically predicted
ultimate curvature for this specimen ($\phi_u = 59 \times 10^{-6}$ /mm) is reached. Similarly, the
displacement ductility capacities of Specimen F2 and F1 ($7\delta_y$ and $8\delta_y$, respectively) are reached

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shortly after the analytically predicted ultimate curvatures for these specimens ($\phi_u = 81 \times 10^{-6}$ 1/mm and $\phi_u = 104 \times 10^{-6}$ 1/mm, respectively) are reached.

From the curvature data and the yield curvature, the curvature ductility was determined at 254 mm (10 in.) above the column footing interface (178 mm (7 in.) for Specimen DV1). The curvature ductility is a ratio of the curvature of the cross-section to the yield curvature:

$$\mu_\phi = \frac{\Phi}{\phi_y}$$

The curvature ductility is plotted vs. the displacement ductility level in Figure 6.14.

In Chapter 3, Equation 3.22 was presented as one approach to estimate the required curvature ductility, $\mu_\phi$, from the required displacement ductility, $\mu_\Delta$ (Priestley and Park 1987):

$$\mu_\Delta = 1 + 3 (\mu_\phi - 1) \frac{L_p}{L} (1 - 0.5 \frac{L_p}{L})$$  \hspace{1cm} (3.22)

where the plastic hinge length is calculated from Equation 3.23:

$$L_p = 0.08 L + 0.022 \frac{1}{\text{MPa}} f_{sy} d_b$$  \hspace{1cm} (3.23)

Equation 3.22 was solved for $\mu_\phi$ and plotted on Figure 6.14. Figure 6.14 indicates that at each displacement ductility level, the experimental curvature ductility is similar to but generally less than the curvature ductility obtained from Equation 3.22. This indicates that for a given curvature ductility, more displacement ductility is available than predicted by Equation 3.22. Finally, in Section 4.5.1, the required curvature ductility of the axial flexural specimens was given as $\mu_\phi =$
4.3, which corresponds to displacement ductility $\mu_a = 2.3$. The point $\mu_a = 4.3, \mu_a = 2.3$ is marked with an "X" on Figure 6.14. This point correlates well with other data in the figure. The results in Figure 6.13 show that analytically estimated curvature is larger than the experimentally determined curvature at displacement ductility $\mu_a = 2.3$. However, Figure 6.14 shows that the analytically predicted curvature ductility is similar to the experimentally determined curvature. The primary reason for this discrepancy is the difference between analytical and experimental yield curvatures (Table 6.3).

**Maximum Extreme Fiber Concrete Compressive Strain**

In Section 4.5.1, the ultimate curvature of the retrofit test specimens was predicted from fiber section analyses that relate curvature to the ultimate concrete compressive strain. This ultimate concrete compressive strain was determined from the \textit{in situ} jacket rupture strain and limiting dilation ratio reported by Kestner et al. (1997) (Equation 4.2):

$$
\varepsilon_{cu} = \frac{\varepsilon_{jr}}{\eta}
$$

Using the jacket rupture strain, $\varepsilon_{jr} = 9000 \, \mu_\varepsilon$, and $\eta$ values as a function of the FRP jacket stiffness from Kestner et al. (1997), the ultimate concrete compressive strains, $\varepsilon_{cu}$, were predicted for each retrofit specimen and given in Table 4.8. Two cases were used to predict the ultimate curvature of the retrofit specimens: (1) when the extreme fiber concrete compressive strain, $\varepsilon_{c,ef}$, reached $\varepsilon_{cu}$; and, (2) when the extreme concrete compressive strain, $\varepsilon_{c,ef}$ reached 1.5 $\varepsilon_{cu}$. In this section, these extreme fiber concrete compressive strains values are compared with experimentally obtained extreme fiber concrete compressive strains at which distress and damage in the test specimens was observed.
The extreme fiber concrete compressive strains at the maximum displacement in each of the first north and south cycles and third north and south cycles were determined by multiplying the curvature, discussed previously, by the neutral axis depth, which was determined from the linear potentiometers:

\[ \epsilon_{c, ef} = \phi \ kd \]  

(6.13)

Four values (first north cycle, first south cycle, third north cycle, and third south cycle) of the maximum extreme fiber maximum concrete compression strain, \( \epsilon_{cm, ef} \), for each displacement ductility level are plotted in Figure 6.15 as open circles. In Figure 6.15, the extreme fiber strains are plotted versus displacement ductility on the bottom axis and curvature ductility on the top axis, where the curvature ductility is calculated from the displacement ductility using Equation 3.22. The strain values have a large scatter at each displacement ductility level. An average of the maximum extreme fiber concrete compressive strains at each displacement ductility level was computed and plotted in Figure 6.15 as a dark line.

Visual observations of distress in the jackets of the retrofit specimens are summarized in Tables 6.5 and 6.6. Points corresponding to visual observations of bulging of the jacket, and to evidence of bulging of the jacket in the jacket strain data (from Figure 6.11) are also shown in Figure 6.15. In addition, observations of longitudinal reinforcing bar buckling, longitudinal reinforcing bar rupture, and jacket rupture are indicated on Figure 6.15. The predicted values of the extreme fiber concrete compressive strain, \( \epsilon_{c, ef} \) at jacket rupture, equal to \( \epsilon_{cu} \) or 1.5 \( \epsilon_{cu} \), are indicated by horizontal lines on Figure 6.15.

The plots of \( \epsilon_{cm, ef} \) vs. displacement ductility show that as the ductility increases, \( \epsilon_{cm, ef} \) also increases. The value of \( \epsilon_{cm, ef} \) at which bulging was visually observed generally increases with the
number of plies in the jacket (i.e., with the jacket stiffness). For the axial flexural specimens, the value of $\varepsilon_{cm,ef}$ where bulging was evident in the jacket strain data is close to, but less than, $\varepsilon_{cm,ef} = \varepsilon_{cu}$. Longitudinal reinforcing bar rupture occurs after the bulging of the jacket is apparent in the jacket strain data. For Specimen F2, the value of $\varepsilon_{cm,ef}$ corresponding to longitudinal reinforcing bar rupture lies between $\varepsilon_{cu}$ and $1.5 \varepsilon_{cu}$. For Specimen F1, data for $\varepsilon_{cm,ef}$ is not available at the displacement ductility levels when the longitudinal bar ruptures because the potentiometer pins on the west face fell off the column. However, by extrapolating the plot of $\varepsilon_{cm,ef}$ vs. displacement ductility, the value of $\varepsilon_{cm,ef}$ corresponding to longitudinal reinforcing bar rupture lies between $\varepsilon_{cu}$ and $1.5 \varepsilon_{cu}$.

The average values of $\varepsilon_{cm,ef}$ for the test specimens are compared in Figure 6.16. This figure indicates a decrease in $\varepsilon_{cm,ef}$ with an increase in the number of jacket plies. The increase in the number of plies corresponds to an increase in jacket stiffness and therefore a decrease in dilation ratio (Kestner et al. 1997). Figure 4.9 shows a plot of extreme fiber compressive strain vs. curvature, which is similar to the plot of $\varepsilon_{cm,ef}$ vs. displacement ductility graph since the ductility is directly correlated to the curvature. Examination of Figure 4.9 and Figure 6.16 indicates that the trend of the curves is the same, an increase in jacket stiffness provides a decrease in concrete compressive strain at a particular ductility level.

To further illustrate the effect of jacket stiffness, Figure 6.17 shows the jacket strains in Specimens F1, F2, and F3 at a displacement ductility of $6\delta_y$. Jacket strains for the first and third north cycles and the first and third south cycles are shown, except for Specimen F3 because the jacket of Specimen F3 ruptured during the second south cycle. Figure 6.17 shows that the jacket strain at $6\delta_y$ on the compression faces decreases with increasing jacket stiffness. For example, Specimen F1 has the stiffest (6 ply) jacket and has the least jacket strain. The decrease in jacket
strain with increasing jacket stiffness is large enough to provide evidence of a corresponding decrease in concrete compressive strain.

**Extreme Fiber Concrete Compressive Strain Increment**

Reversed cyclic lateral loading caused flexure cracks to occur on both the north and south faces of the test specimens. Overall the column elongates as the test proceeds and residual tensile deformations are measured by the linear potentiometers at the end of each cycle. A second approach to quantify the extreme concrete compressive strain, by including the residual tensile strains, was considered. The cyclic behavior the extreme fiber strain is shown in Figure 6.18 for the north face of Specimen F1. Displacement ductility levels of $1.5\delta_y$ and $4\delta_y$, are shown. During cycles at low displacement ductility levels, such as $1.5\delta_y$, the residual compression and tension strains at zero lateral load are low. However, during cycles at large displacement ductility levels, a residual tensile strain occurs at the extreme fiber due to the formation of flexural cracks. The extreme fiber concrete compressive strain increment, $\varepsilon_{ci,ef}$ is defined as the sum of the maximum extreme fiber concrete compressive strain and the residual tensile strain.

$$\varepsilon_{ci,ef} = \varepsilon_{cm,ef} + \varepsilon_{cr,ef}$$  \hspace{1cm} (6.14)

The maximum extreme fiber concrete compressive strain and the extreme fiber compressive strain increment are plotted in Figure 6.19 for Specimens F1 and F2. The difference in strain between the two curves is equal to the residual tension strain in the column when the lateral load is zero.

The extreme fiber concrete compressive strain increment, $\varepsilon_{ci,ef}$ is plotted vs. displacement ductility and curvature ductility for each test specimen in Figure 6.20. Four values (first north cycle, first south cycle, third north cycle, and third south cycle) of $\varepsilon_{ci,ef}$ for each displacement ductility level are plotted in Figure 6.20 as open circles. The strain values have a large scatter at
a particular ductility level. An average of the extreme fiber concrete compressive strain increments at each displacement ductility level was computed and is shown as a dark line in Figure 6.20. Points corresponding to visual observations of bulging of the jacket, and to evidence of bulging of the jacket in the jacket strain data are also shown in Figure 6.20. In addition, longitudinal reinforcing bar buckling, longitudinal reinforcing bar rupture, and jacket rupture are also indicated on Figure 6.20. The predicted value of the extreme fiber concrete compressive strain, $\varepsilon_{c_{uf}}$, at jacket rupture, equal to $\varepsilon_{c_{up}}$ or $1.5 \varepsilon_{c_{up}}$, are indicated by horizontal lines on Figure 6.20.

**Experimental vs. Analytical Concrete Strain at Failure**

In Section 4.5.1, the extreme fiber concrete compressive strain, $\varepsilon_{c_{m,ef}}$, at failure of the retrofit specimens was expected to be between $\varepsilon_{c_{uu}}$ and $1.5 \varepsilon_{c_{uu}}$. Figure 6.15 indicates that when $\varepsilon_{c_{m,ef}}$ is in the range (between $\varepsilon_{c_{uu,ef}}$ and $1.5 \varepsilon_{c_{uu,ef}}$) distress in the retrofit test specimens is observed. Table 6.7 compares the analytically predicted displacement ductility when $\varepsilon_{c_{m,ef}} = \varepsilon_{c_{uu}}$ and $1.5 \varepsilon_{c_{uu}}$, and the experimentally observed displacement ductility corresponding to the sustainable lateral load carrying capacity (Tables 5.1 and 5.2).

The analytically predicted displacement ductilities are obtained by dividing the curvature data in Table 4.8 (for $f_{\text{residual}} = 5 \text{ MPa}$) by the analytical yield curvature (17 X 10$^{-6}$ 1/ mm) to obtain curvature ductility, and then applying Equation 3.22 with $f_y$ equal to the minimum specified yield stress of the longitudinal reinforcing steel (414 MPa (60 ksi)). The failure mode of the retrofit specimens considered in Section 4.5.1 was jacket rupture. Specimen F3 was the only specimen that failed by jacket rupture. Although the jackets of Specimens F1 and F2 did not rupture, the ultimate concrete compressive strain corresponding to the jacket rupture strain, $\varepsilon_y = 9000 \mu \varepsilon$, and the limiting dilation ratio, $\eta$, from Kestner et al. (1997) (i.e. $\varepsilon_{c_{uu}} = \varepsilon_y / \eta$) appears to be an
indication of specimen failure. When $e_u$ is reached, the jacket begins to bulge significantly. The bulging can lead to one of two failure modes of the column:

1. The transverse strains in the jacket are close to the rupture strain, so that jacket rupture occurs with continued loading;

2. The support of the longitudinal reinforcing bar at the face of the column is decreasing, so that the bar begins to buckle and will rupture in subsequent cycles.

Ductile Shear vs. Axial Flexural Specimen Concrete Compression Strain Data

The average values of $e_{cm, ef}$ are plotted against the displacement ductility in Figure 6.16 for all the retrofit specimens. Specimens DV2 and F2, with four ply jackets, can be compared. These comparisons indicate that the values of $e_{cm, ef}$ for DV2 are lower than the values of $e_{cm, ef}$ for F2. The smaller values of $e_{cm, ef}$ for Specimen DV2 occur for two reasons. First, the moment gradient is steeper in DV2. The linear potentiometer gage length used to determine curvatures and concrete strains was the same for the ductile shear and the axial flexural specimens, and as a result the moment and average curvature over this gage length is smaller for the ductile shear specimens. Second, the displacement of the column includes both flexure and shear deformation of the column. The higher shear forces in the ductile specimens produce more shear deformation than in the axial flexural specimens. Shear cracks in the retrofit ductile shear specimens were found when the jacket was removed after the test. Therefore, at each displacement ductility level, less flexural deformation is required in the ductile shear specimens because of the increased shear deformation. The decreased curvature in the ductile shear specimens corresponds to a decreased extreme fiber concrete compressive strain increment.
Experimental vs. Analytical Jacket Strains at Failure

Figure 6.11 shows that the measured jacket strains range from 4000 to 6000 με during cycles to the displacement ductility level corresponding to the maximum sustainable lateral load (80% of the maximum load). Thus, as the retrofit specimens approach failure, the measured jacket strains are well below the jacket rupture strain of 9000 με assumed in the analyses. The difference in strains at failure occurs for two reasons: (1) the jacket strains were not measured at the corners which is the most critical location (e.g., the jacket on Specimen F3 ruptured at the corner); and (2) most of the specimens failed by rupture of the longitudinal reinforcing bars. Although the jackets of Specimens F1, F2, DV1 and DV2 did not rupture before longitudinal reinforcing bar rupture, the ultimate concrete compressive strain corresponding to (i.e., $\epsilon_{cu} = \epsilon_{jr}$ / $\eta$) the jacket rupture strain, $\epsilon_{jr} = 9000$ με, and the limiting dilation ratio, $\eta$, from Kestner et al. (1997) appears to be an indication of specimen failure.

6.3.2 Shear Response

The shear strength of the non-retrofit and retrofit test specimens was discussed in Chapter 4. The shear strength of the test specimens was predicted using a model proposed by Priestley et al. (1994). In this section, the shear demands on the test specimens will be compared with the predicted shear strengths.

Shear Resistance of Specimen DV0

The shear resistance of Specimen DV0 was calculated using a model proposed by Priestley et al. (1994) in Section 4.5.2. The model includes three components to shear resistance (Equation 3.25):

$$V_n = V_c + V_s + V_p$$
The shear resistance provided by the axial load, $V_p'$, was neglected. The resistance provided by the transverse reinforcing steel was 63 kN (14.1 kips) when calculated using the minimum specified yield stress, of 414 MPa (60 ksi). When the actual yield stress of 438 kN (63.6 ksi) is used in the calculation, $V_s = 67$ kN (15.1 kips). The shear resistance provided by the concrete, $V_e$, decreases with increased displacement ductility, $\mu_d$. The shear resistance of the concrete is:

$$V_c = k \sqrt{f'_c} 0.8 A_g$$

(3.9)

The coefficient $k$ varies linearly from 0.3 for $\mu_d \leq 2$ to 0 for $\mu_d \geq 4$. At $\mu_d = 2$, $V_c = 264$ kN (59.4 kips) using the specified concrete compressive strength. When the actual concrete compressive strength of 26.5 MPa (3843 psi) is used $V_c = 259$ kN (55.2 kips) at $\mu_d = 2$. According to the model the total shear resistance ($V_c + V_s = 259$ kN + 67 kN) is 326 kN (73.3 kips). At $\mu_d = 4$, the total shear resistance is 67 kN (15.1 kips). These results are plotted in Figure 6.21 (a).

The test results for Specimen DV0 are plotted in Figure 6.21 (a) as three data points corresponding to the shear resistance at displacements of $\delta_y$, $2\delta_y$, and $2.5\delta_y$. Figure 6.21 (a) shows that the shear resisted by Specimen DV0 was close to the shear strength predicted by the model at the peak shear of 320 kN (71.9 kips). Specimen DV0 failed in shear several cycles after the peak shear was reached as a result of the degradation of shear resistance with increased displacement ductility. The rate of shear degradation with displacement ductility in Specimen DV0 was faster than the shear strength degradation predicted by the model. Thus, Specimen DV0 failed just after the peak at a displacement ductility level of $2.5\delta_y$.

Shear Resistance of Retrofit Ductile Shear Specimens

The retrofit of the ductile shear specimens, DV1 and DV2, was considered successful because shear failure was prevented and the specimens failed in flexure after significant
displacement ductility levels were reached. Since Specimens DV1 and DV2 did not fail in shear, their shear strength cannot be obtained from the test results. However, the shear resisted by Specimen DV1, which had a two ply jacket, provides some information on the shear capacity of a jacketed specimen.

As explained in Section 6.1.6, increases in displacement decrease the shear demand on the test specimens (as well as on the columns in a building) because of the P-Δ effect. Figure 6.21 (b) shows the shear resisted by Specimen DV1 at each displacement ductility level. The shear force shown in Figure 6.21 (b) is the shear at maximum displacement in the first north cycles. As shown, the maximum shear is 302 kN (67.9 kips). The shear force decreases with increasing displacement ductility because the shear demand decreases.

In Section 4.5.2, the shear strength of the retrofit specimens was investigated for two jacket strains: 4000 με, the strain at which the concrete shear resistance component, V_e, is expected to be lost (Seible et al. 1997); and, 9000 με, the anticipated jacket rupture strain. At a strain of 4000 με, the shear that would be resisted by the jacket of DV1 alone, V_j, is 278 kN (62.5 kips), at a strain of 9000 με, V_j is 626 kN (140.7 kips). In addition, the shear resistance of the transverse reinforcing steel, V_s, is 67 kN (15.1 kips).

Assuming the concrete shear resistance component is negligible, the total shear resistance of the jacket and the transverse reinforcing steel would be 345 kN (77.6 kips) at a jacket strain of 4000 με, and 693 kN (155.8 kips) at a jacket strain of 9000 με. The shear demand on Specimen DV1 is approximately 300 kN (67.5 kips) at a displacement ductility of μΔ = 4, and because the jacket strains at the center of the east and west faces of Specimen DV1 are below 4000 με at μΔ = 4 (Figure 6.11), it appears that the concrete shear resistance remains considerable and the jacket is not being fully mobilized in shear.
The most critical displacement ductility level for Specimen DV1 is in the range of 2 to $4\delta_y$. At these ductilities, the hinge region has formed and concrete shear resistance, $V_e$, is expected to degrade form flexural ductility demands, while the displacement ductility level is not yet high enough for the P-Δ effect to significantly decrease the shear demand. The maximum ductility required for seismic design will often exceed 2 to $4\delta_y$. Thus the retrofit of a ductile shear column, such as DV1, requires a jacket designed for flexural ductility. In the case of Specimen DV1, the flexural retrofit was sufficient to confine the concrete and maintain the concrete shear resistance. In addition, the jacket contributed additional shear resistance. The degradation of the concrete shear resistance occurs only in the plastic hinge region, and thus the length of the jacket required to retrofit a ductile shear column can be the same as that required for axial flexural retrofit, which covers the plastic hinge region. Thus, for columns similar to the non-ductile reinforced concrete columns studied in this investigation, it appears that the axial flexural retrofit is sufficient to retrofit ductile shear columns.

6.4 SUMMARY

The test results for the non-retrofit and retrofit specimens are summarized as follows:

(1) The non-retrofit test specimens failed as expected. The axial flexure test specimen (Specimen F0) failed in the axial flexural mode, with spalling of the cover, crushing of concrete, and buckling of longitudinal reinforcing steel in compression at a displacement ductility of $2\delta_y$. The ductile shear specimen (Specimen DV0) failed in shear after yielding in flexure and reaching a displacement ductility of $2\delta_y$.

(2) The ductility of the retrofit test specimens was limited by two different failure modes: (1) rupture of the jacket, observed in Specimen F3; and (2) rupture of the longitudinal reinforcing bars as a result of low cycle fatigue from buckling and straightening of the
bars, observed in Specimens F1, F2, DV1 and DV2. Specimen F3 was the only specimen for which the ductility was controlled by jacket strength. Thus, it appears that when the jacket strength is sufficient to prevent jacket rupture, stiffness is the jacket property that influences the ductility of the retrofit specimen. Although the displacement ductility increases with the stiffness of the jacket (i.e., the number of plies of carbon tow sheet), the maximum displacement ductility which can be achieved with a FRP jacket retrofit on a square reinforced concrete column is limited by buckling and subsequent rupture of the longitudinal reinforcing bars, which is not significantly influenced by increasing the jacket stiffness.

(3) The flexural strength of the axial flexural test specimens was approximately 25% larger than predicted analytically. This was partly due to the 11% overstrength of the longitudinal reinforcing steel. The flexural strength of the retrofit specimens was similar to the flexural strength of the non-retrofit specimens. Therefore, the flexural strength was not significantly increased by the presence or stiffness of the jackets.

(4) The shear strength of the non-retrofit ductile shear test specimens was close to the shear strength predicted by the model used in Chapter 4. This model adds the shear resistance of the transverse reinforcing steel, $V_s$, to the shear resistance of the concrete, $V_c$, which degrades with increasing flexural ductility levels (Priestley et al. 1994).

(5) The behavior of the retrofit ductile shear specimens was significantly improved by the jacket retrofits, including one jacket with only two plies of carbon fiber tow sheet. The jackets maintained the shear resistance of the concrete, and provided additional shear resistance from stress developed in the jacket. As a result, the ductile shear failure mode was eliminated. To provide the required levels of displacement ductility, the jackets for
the ductile shear specimens were designed considering axial flexural failure. The retrofit ductile shear specimens failed in the axial flexural mode, but provided the required ductility. For columns similar to the test specimens, it appears that a retrofit against the axial flexural failure mode is sufficient to retrofit against the ductile shear failure mode.

(6) The retrofit specimens had approximately the same stiffness as the non-retrofit specimens. The initial stiffness of the test specimens could be estimated analytically using a moment of inertia between the gross moment of inertia, $I_p$, and the effective moment of inertia based on the ACI 318-95 code, $I_{e,ACI}$.

(7) The energy dissipation per half cycle of the test specimens increased with increasing displacement ductility level until failure. The energy dissipation per half cycle of the retrofit specimens was not significantly influenced by the presence or properties of the jackets. Failure of the retrofit specimens was not preceded by deterioration in the hysteretic behavior of the specimens, and a significant decrease in the energy dissipation per half cycle was not observed before failure.

(8) The experimentally determined yield curvature of the test specimens was less than (i.e, about half) the analytically predicted yield curvature. Similarly, the experimentally determined curvatures of the retrofit specimens at a given displacement ductility level were less than the analytically predicted curvatures. As a result, when the experimentally determined curvatures reached the analytically predicted ultimate curvatures for the retrofit specimens, the corresponding displacement ductility was greater than predicted analytically.

(9) The analytically predicted ultimate curvatures were reached in the tests of the retrofit specimens just before the displacement ductility capacity was reached. Thus, the
analytically predicted ultimate curvature provides an indication of the displacement ductility capacity.

(10) The experimentally determined extreme fiber compression strains of the retrofit specimens at a given displacement ductility level were less than the strains expected from the fiber cross-section analyses discussed in Chapter 4. As a result, when the experimentally determined extreme fiber compression strains reached the analytically predicted ultimate concrete compressive strains for the retrofit specimens, the corresponding displacement ductility was greater than predicted analytically.

(11) In the tests of the retrofit specimens, the extreme fiber concrete compression strains reached the analytically predicted ultimate concrete compression strain just before the displacement ductility capacity was reached. That is, the ultimate compression strain at the extreme fiber provides an indication of the displacement ductility capacity. It is noted, however, that the analytically predicted ultimate concrete compressive strains (and the corresponding ultimate curvatures) correspond to rupture of the jackets of the retrofit specimens, while most of the failures of the retrofit specimens involved buckling and rupture of the longitudinal reinforcing bars.

(12) The experimentally determined displacement ductility capacity of the retrofit specimens was significantly greater than the displacement ductility capacity of the non-retrofit specimens. The displacement ductility capacity of the retrofit specimens increased with the stiffness (i.e., the number of plies) of the jackets. A jacket with as few as two plies of carbon fiber tow sheet significantly increased the ductility capacity. The experimentally determined displacement ductility capacity of the retrofit specimens was greater than expected from the analytical results discussed in Chapter 4.
The jacket strain increases with displacement ductility level and with cyclic loading at a single ductility level. Eventually, the jacket begins to bulge, and this bulging was observed visually and also appears as an increase in jacket strain where the bulging occurs.
Table 6.1 Column stiffness: experimental and theoretical.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Experimental</th>
<th>Analytical: $K = 3 E_c I_e / L^3$, kN/mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>cycle</td>
<td>$K_e$, kN/mm</td>
</tr>
<tr>
<td>F0</td>
<td>1st</td>
<td>15.1</td>
</tr>
<tr>
<td></td>
<td>3rd</td>
<td>11.2</td>
</tr>
<tr>
<td>F1</td>
<td>1st</td>
<td>13.8</td>
</tr>
<tr>
<td></td>
<td>3rd</td>
<td>9.3</td>
</tr>
<tr>
<td>F2</td>
<td>1st</td>
<td>12.3</td>
</tr>
<tr>
<td></td>
<td>3rd</td>
<td>9.4</td>
</tr>
<tr>
<td>F3</td>
<td>1st</td>
<td>15.9</td>
</tr>
<tr>
<td></td>
<td>3rd</td>
<td>11.4</td>
</tr>
<tr>
<td>DV0</td>
<td>1st</td>
<td>43.8</td>
</tr>
<tr>
<td></td>
<td>3rd</td>
<td>31.3</td>
</tr>
<tr>
<td>DV1</td>
<td>1st</td>
<td>43.8</td>
</tr>
<tr>
<td></td>
<td>3rd</td>
<td>37.3</td>
</tr>
<tr>
<td>DV2</td>
<td>1st</td>
<td>47.3</td>
</tr>
<tr>
<td></td>
<td>3rd</td>
<td>35.0</td>
</tr>
</tbody>
</table>
Table 6.2 Observed and calculated cracking moment.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>number of plies, n</th>
<th>cracking moment at interface, kN m</th>
<th>location of second transverse crack, mm</th>
<th>corresponding cracking moment, kN m</th>
<th>calculated cracking moment, kN m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>S</td>
<td>N</td>
<td>S</td>
</tr>
<tr>
<td>F0</td>
<td>0</td>
<td>152</td>
<td>175</td>
<td>279</td>
<td>163</td>
</tr>
<tr>
<td>F1</td>
<td>6</td>
<td>239</td>
<td>178</td>
<td>171</td>
<td>248</td>
</tr>
<tr>
<td>F2</td>
<td>4</td>
<td>206</td>
<td>178</td>
<td>178</td>
<td>215</td>
</tr>
<tr>
<td>F3</td>
<td>2</td>
<td>163</td>
<td>184</td>
<td>229</td>
<td>229</td>
</tr>
<tr>
<td>DV0</td>
<td>0</td>
<td>163</td>
<td>152</td>
<td>279</td>
<td>180</td>
</tr>
<tr>
<td>DV1</td>
<td>4</td>
<td>192</td>
<td>178</td>
<td>191</td>
<td>224</td>
</tr>
<tr>
<td>DV2</td>
<td>2</td>
<td>149</td>
<td>165</td>
<td>197</td>
<td>240</td>
</tr>
</tbody>
</table>

Table 6.3 Analytical and experimental yield curvature results.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Experimental, 1/mm</th>
<th>Analytical, 1/mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>F0</td>
<td>$10 \times 10^{-6}$</td>
<td></td>
</tr>
<tr>
<td>F1</td>
<td>$8 \times 10^{-6}$</td>
<td></td>
</tr>
<tr>
<td>F2</td>
<td>$8 \times 10^{-6}$</td>
<td></td>
</tr>
<tr>
<td>F3</td>
<td>$10 \times 10^{-6}$</td>
<td></td>
</tr>
<tr>
<td>DV0</td>
<td>$12 \times 10^{-6}$</td>
<td></td>
</tr>
<tr>
<td>DV1</td>
<td>$8 \times 10^{-6}$</td>
<td></td>
</tr>
<tr>
<td>DV2</td>
<td>$9 \times 10^{-6}$</td>
<td>$17 \times 10^{-6}$</td>
</tr>
</tbody>
</table>
Table 6.4 Axial flexural and ductile shear specimen curvature results.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Curvature, 10^{-6}/mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>at 254 mm from the column footing interface.</td>
</tr>
<tr>
<td></td>
<td>2\delta_y</td>
</tr>
<tr>
<td>cycle</td>
<td>N</td>
</tr>
<tr>
<td>F0</td>
<td>28.6</td>
</tr>
<tr>
<td>F1</td>
<td>10.3</td>
</tr>
<tr>
<td>F2</td>
<td>17.2</td>
</tr>
<tr>
<td>F3</td>
<td>24.4</td>
</tr>
<tr>
<td>DV0</td>
<td>13.9</td>
</tr>
<tr>
<td>DV1*</td>
<td>10.8</td>
</tr>
<tr>
<td>DV2</td>
<td>8.2</td>
</tr>
</tbody>
</table>

* Specimen DV1 curvature data is reported for a location 178 mm above the column footing interface.
N/O = not observed
N/A = not available
Table 6.5 Location and displacement ductility at which bulging of jacket occurred.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Number of plies, n</th>
<th>First occurrence of bulging</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>μ⁻¹</td>
<td>location, mm</td>
<td>jacket strain, με</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>N</td>
</tr>
<tr>
<td>F1</td>
<td>6</td>
<td>5</td>
<td>51</td>
<td>1371</td>
</tr>
<tr>
<td>F2</td>
<td>4</td>
<td>3</td>
<td>102</td>
<td>1773</td>
</tr>
<tr>
<td>F3</td>
<td>2</td>
<td>2.5</td>
<td>127</td>
<td>3736</td>
</tr>
<tr>
<td>DV1</td>
<td>2</td>
<td>4</td>
<td>178</td>
<td>2175</td>
</tr>
<tr>
<td>DV2</td>
<td>4</td>
<td>4</td>
<td>100</td>
<td>1395</td>
</tr>
</tbody>
</table>

Location of bulging and buckling was measured from the column footing interface. Data was obtained for the north cycles.

Table 6.6 Location and displacement ductility at which buckling of jacket occurred.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Number of plies, n</th>
<th>First occurrence of buckling</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>μ⁻¹</td>
<td>location (mm)</td>
<td>jacket strain, με</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>N</td>
</tr>
<tr>
<td>F1</td>
<td>6</td>
<td>N/O</td>
<td>N/O</td>
<td>N/O</td>
</tr>
<tr>
<td>F2</td>
<td>4</td>
<td>5</td>
<td>380</td>
<td>3476</td>
</tr>
<tr>
<td>F3</td>
<td>2</td>
<td>2.5</td>
<td>127 / 229</td>
<td>3736</td>
</tr>
<tr>
<td>DV1</td>
<td>2</td>
<td>4</td>
<td>178</td>
<td>2175</td>
</tr>
<tr>
<td>DV2</td>
<td>4</td>
<td>5</td>
<td>380</td>
<td>1986</td>
</tr>
</tbody>
</table>

Location of bulging and buckling was measured from the column footing interface. Data was obtained for the north cycles. N/O = not observed
Table 6.7 Analytical and experimental ultimate displacement ductility results.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Experimental</th>
<th>Analytical</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>displacement ductility ( \cdot )</td>
</tr>
<tr>
<td>F0</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>F1</td>
<td>8</td>
<td>3.4</td>
</tr>
<tr>
<td>F2</td>
<td>7</td>
<td>2.8</td>
</tr>
<tr>
<td>F3</td>
<td>5</td>
<td>2.2</td>
</tr>
<tr>
<td>DV0</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>DV1</td>
<td>7</td>
<td>2.5</td>
</tr>
<tr>
<td>DV2</td>
<td>8</td>
<td>3.3</td>
</tr>
</tbody>
</table>

\( \cdot \) displacement ductility using \( \varepsilon_{c, ef} = \varepsilon_{cu} \).

\( \dagger \) displacement ductility using \( \varepsilon_{c, ef} = 1.5 \varepsilon_{cu} \).
Figure 6.1 Backbone load vs. displacement curves.
Figure 6.2 Stiffness of the non-retrofit and retrofit columns at different displacement ductilities.
Figure 6.3 Displacement ductility vs. number of jacket plies.
Figure 6.4 Specimen F2 hysteresis with elastic energy and energy per half cycle shown.
Fig. 6.5 (a) Normalized energy per half cycle for axial flexural specimens.

Specimens F1, F2, and F3 were not subjected to additional cycles at $1.5\delta$. 
Figure 6.5 (b) Normalized energy per half cycle for ductile shear specimens.
Figure 6.6 Crack patterns in non-retrofit specimens.
Figure 6.6 Crack patterns in non-retrofit specimens.
Specimen DV0
Figure 6.7 Loading conditions for building column and test specimen.
Figure 6.8 Displacement to height ratios with increased displacement ductility.
Figure 6.9 Curvature data from potentiometers.

(a) Curvature for Specimen F0 from west face potentiometers.

(b) Curvature for Specimen F0 from north and south face potentiometers.
Curvature data from potentiometers.

(c) Curvature for Specimen F1 from west face potentiometers.

(d) Curvature for Specimen F1 from north and south face potentiometers.
Figure 6.9 (continued) Curvature data from potentiometers.

(e) Curvature for Specimen F2 from west face potentiometers.

(f) Curvature for Specimen F2 from north and south face potentiometers.
(g) Curvature for Specimen F3 from west face potentiometers.

(h) Curvature for Specimen F3 from north and south face potentiometers.
(i) Curvature for Specimen DV0 from west face potentiometers.

(j) Curvature for Specimen DV0 from north and south face potentiometers.
(k) Curvature for Specimen DV1 from west face potentiometers.

(l) Curvature for Specimen DV1 from north and south face potentiometers.
(m) Curvature for Specimen DV2 from west face potentiometers.

(n) Curvature for Specimen DV2 from north and south face potentiometers.
Figure 6.10 Jacket strain gage locations.
Specimen F1 cycled in south direction only after 88,.

Indicates bulging at 229 mm from the column footing interface.

(a) Specimen F1
Figure 6.11 (continued): Jacket strains, µε.

South face indicates bulging at 299 mm from the column footing interface.

- Compression face
- East face
- North tension face
- West face

(b) Specimen F2

Δ = questionable data
◆ = questionable data near end of test
Figure 6.11 (continued) Jacket strains, %e.

(c) Specimen F3

- South compression face
- East
- North tension face
- West

jacket rupture located at southeast corner at 6o.

= questionable data
= questionable data near end of test
Figure 6.11 (continued) Jacket strains, με.

(d) Specimen DV1

- = questionable data near end of test

South compression face  East

North tension face  West
Figure 6.11 (continued). Jacket strains, με.

(e) Specimen DV2
Figure 6.12 Jacket behavior with combined axial and reversed cyclic lateral loading.

- (a) tension face
- (b) side view
  - flexural cracks
  - bulging and buckling of jacket
- (c) compression face
- (d) damaged concrete
  - reduced confinement near compression face
  - loss of lateral support to longitudinal bars leading to bar rupture
- (e) loss of confinement and restraint to longitudinal reinforcing bar.
Figure 6.12 Jacket behavior with combined axial and reversed cyclic lateral loading.
Figure 6.13 Ultimate curvature vs. displacement ductility.
Figure 6. 14 Comparison of analytical and experimental values for curvature ductility, $\mu_\theta$. 
Figure 6.15 Maximum concrete compressive strains, $\varepsilon_{cm, ef}$, at displacement ductility levels.

(a) Specimen F0
- Observed bulging
- Bulging evident from jacket strain data
- Longitudinal reinforcing bar rupture

(b) Specimen F1
- Observed bulging
- Bulging evident from jacket strain data
- Longitudinal reinforcing bar rupture

Strain data not available

Displacement ductility $\mu_d$ vs. curvature ductility $\mu_b$

Displacement ductility $\mu_d$ vs. maximum concrete strain at extreme fiber $\varepsilon_{cm, ef}$

- $\varepsilon_{cm, ef} = 1.5 \varepsilon_{cu} = 27000$
- $\varepsilon_{cm, ef} = \varepsilon_{cu} = 18000$
- $\varepsilon_{cm, ef} = 16000$
- $\varepsilon_{cm, ef} = 7000$

Curvature ductility $\mu_b$ vs. displacement ductility $\mu_d$
Figure 6.15 (continued) Maximum concrete compressive strains, $\varepsilon_{cm, ef}$ at displacement ductility levels.
Figure 6.15 (continued) Maximum concrete compressive strains, $\varepsilon_{cm, ef}$, at displacement ductility levels.
Figure 6.15 (continued) Maximum concrete compressive strains, $\varepsilon_{\text{cm, ef}}$, at displacement ductility levels.
Figure 6.16 Comparison of maximum concrete compressive strain, $\varepsilon_{cm,st}$, at displacement ductility levels for axial flexural and ductile shear specimens.
Figure 6.17: Jacket strains at 60, for axial flexural specimens, με.

- South compression face
- East
- North tension face
- West

- Specimen F3 Jacket Rupture

- ✦ = questionable data
Figure 6.18 Strain at extreme fiber on north face of the column throughout load cycle.

extreme fiber strain on north face of Specimen F1 $\varepsilon_{ef}$
Figure 6.19 Residual strains, $\varepsilon_{\text{cr, ef}}$, at displacement ductility levels.
Figure 6.20 Concrete compressive strain increment, $\varepsilon_{ci,cr}$, at displacement ductility levels.
Figure 6.20 (continued) Concrete compressive strain increment, $\varepsilon_{ci,ef}$, at displacement ductility levels.

- $\varepsilon_{ci,ef} = 43000$
- $\varepsilon_{ci,ef} = 1.5 \varepsilon_{cu} = 27000$
- $\varepsilon_{ci,ef} = \varepsilon_{cu} = 18000$
- $\varepsilon_{ci,ef} = 15000$

- observed bulging
- bulging evident from jacket strain data
- longitudinal reinforcing bar rupture

(b) Specimen F1

Displacement ductility $\mu_d$

Curvature ductility $\mu_q$

Strain data not available
Figure 6.20 (continued) Concrete compressive strain increment, $\varepsilon_{ci, ef}$, at displacement ductility levels.
Figure 6.20 (continued) Concrete compressive strain increment, $\varepsilon_{ci,ef}$ at displacement ductility levels.
Figure 6.20 (continued) Concrete compressive strain increment, $\varepsilon_{ci,ef}$ at displacement ductility levels.
curvature ductility $\mu_s$

(g) Specimen DV2

- observed bulging
- $\bigcirc$ longitudinal reinforcing bar rupture

$\varepsilon_{cl, ef} = 1.5 \varepsilon_{cu} = 24000$

$\varepsilon_{cl, ef} = \varepsilon_{cu} = 16000$

$\varepsilon_{cl, ef} = 6000$

Figure 6.20 (continued) Concrete compressive strain increment, $\varepsilon_{cl, ef}$, at displacement ductility levels.

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(a) Shear force in non-retrofit specimen DV0 compared to shear strength model.

(b) shear demand in Specimen DV1

Figure 6.21 Shear force at displacement ductility levels.
CHAPTER 7
SUMMARY, CONCLUSIONS, AND FUTURE RESEARCH

7.1 SUMMARY OF INVESTIGATION

This report presents the results of an investigation of the behavior of square non-ductile reinforced concrete building columns retrofit with fiber reinforced polymer (FRP) jackets. As a result of poor reinforcing details, non-ductile reinforced concrete building columns have four potential failure modes: axial flexural failure, ductile shear failure, brittle shear failure, and lap splice failure. Each of these failure modes is discussed in Chapter 2. Current work at Lehigh University addresses retrofit for against these four failure modes. This investigation focuses on retrofit against axial flexural failures and ductile shear failures. The objectives of this investigation were:

(1) To investigate the use of FRP jackets as a retrofit for axial flexural and ductile shear failures in non-ductile reinforced concrete building columns.

(2) To evaluate analytical methods and design guidelines for the retrofit of non-ductile reinforced concrete building columns using FRP jackets.

Full-scale retrofit and non-retrofit column test specimens were tested under combined axial and reversed cyclic lateral loads. The FRP jackets were made from carbon fiber tow sheet applied with epoxy resin on existing reinforced concrete test specimens.

The strength and stiffness of the jackets, as controlled by the number of plies of carbon fiber tow sheet in the jackets, and the expected failure mode of the non-retrofit columns were the primary variables of the investigation. The jackets used to retrofit the test specimens were designed to increase the flexural ductility of the column specimens without increasing the flexural strength or stiffness. Seven specimens were analyzed and tested. Existing design guidelines for
FRP jacket retrofit (Seible et al. 1995) were used to design some of the retrofits. Other designs were based on fiber cross-section analyses. Fiber section analyses, as well as an existing model for the shear strength of reinforced concrete columns (Priestley et al. 1994) were used to predict the behavior of the retrofit specimens. Expected in situ material properties for the carbon FRP jackets (Kestner et al. 1997), rather than those reported by the manufacturer, were used for the design and analysis of the retrofits. Analytical and experimental results for the strength, stiffness, ductility, energy dissipation, jacket strains, curvatures, and concrete strains for the non-retrofit and retrofit test specimens were reported and discussed. The limit states corresponding to failures of the non-retrofit and retrofit specimens were determined. The results from fiber section analyses and from the shear strength model were compared with the observed response of the test specimens.

7.2 SUMMARY OF FINDINGS

The findings of the investigation are summarized as follows:

1. The non-retrofit axial flexure test specimen (Specimen F0) failed in the axial flexural mode, with spalling of the cover, crushing of concrete, and buckling of longitudinal reinforcing steel in compression at a displacement ductility of $2\delta_y$. The non-retrofit ductile shear specimen (Specimen DV0) failed in shear after yielding in flexure and reaching a displacement ductility of $2\delta_y$.

2. The ductility of the retrofit test specimens was limited by two different failure modes: (1) rupture of the jacket, observed in Specimen F3; and (2) rupture of the longitudinal reinforcing bars as a result of low cycle fatigue from buckling and straightening of the bars, observed in Specimens F1, F2, DV1 and DV2. Specimen F3 was the only specimen for which the ductility was controlled by jacket strength. Thus, it appears that when the
jacket strength is sufficient to prevent jacket rupture, stiffness is the jacket property that influences the ductility of the retrofit specimen. Although the displacement ductility increases with the stiffness of the jacket (i.e., the number of plies of carbon tow sheet), the maximum displacement ductility which can be achieved with a FRP jacket retrofit on a square reinforced concrete column is limited by buckling and subsequent rupture of the longitudinal reinforcing bars, which is not significantly influenced by increasing the jacket stiffness.

(3) The flexural strength of the axial flexural test specimens was approximately 25% larger than predicted analytically. This was partly due to the 11% overstrength of the longitudinal reinforcing steel. The flexural strength of the retrofit specimens was not significantly increased by the presence or stiffness of the jackets.

(4) The shear strength of the non-retrofit ductile shear test specimens was close to the shear strength predicted by adding the shear resistance of the transverse reinforcing steel, $V_s$, to the shear resistance of the concrete, $V_c$, which degrades with increasing flexural ductility levels (Priestley et al. 1994).

(5) The behavior of the retrofit ductile shear specimens was significantly improved by the jacket retrofits, including one jacket with only two plies of carbon fiber tow sheet. As a result, the ductile shear failure mode was eliminated. To provide the required levels of displacement ductility, the jackets for the ductile shear specimens were designed considering axial flexural failure. The retrofit ductile shear specimens, failed in the axial flexural mode, but provided the required ductility. For columns similar to the test specimens, it appears that a retrofit against the axial flexural failure mode is sufficient to retrofit against the ductile shear failure mode.
(6) The retrofit specimens had approximately the same stiffness as the non-retrofit specimens. The initial stiffness of the test specimens could be estimated analytically using a moment of inertia between the gross moment of inertia, $I_g$, and the effective moment of inertia based on the ACI 318-95 code, $I_{ACI}$.

(7) The energy dissipation per half cycle of the test specimens increased with increasing displacement ductility level until failure. The energy dissipation per half cycle of the retrofit specimens was not significantly influenced by the presence or properties of the jackets. Failure of the retrofit specimens was not preceded by deterioration in the hysteretic behavior of the specimens, and a significant decrease in the energy dissipation per half cycle was not observed before failure.

(8) The experimentally determined yield curvature of the test specimens was less than predicted analytically. Similarly, the experimentally determined curvatures of the retrofit specimens at a given displacement ductility level were less than predicted analytically. As a result, when the experimentally determined curvatures reached the analytically predicted ultimate curvatures for the retrofit specimens, the corresponding displacement ductility was greater than predicted.

(9) The analytically predicted ultimate curvatures were reached in the tests of the retrofit specimens just before the displacement ductility capacity was reached. Thus, the analytically predicted ultimate curvatures provide an indication of the displacement ductility capacity.

(10) The experimentally determined extreme fiber compression strains of the retrofit specimens at a given displacement ductility level were less than the strains expected from the fiber cross-section analyses. As a result, when the experimentally determined extreme
fiber compression strains reached the analytically predicted ultimate concrete compressive strains for the retrofit specimens, the corresponding displacement ductility was greater than predicted analytically.

(11) In the tests of the retrofit specimens, the extreme fiber concrete compression strains reached the analytically predicted ultimate concrete compression strain just before the displacement ductility capacity was reached. That is, the ultimate compression strain at the extreme fiber provides an indication of the displacement ductility capacity. It is noted, however, that the analytically predicted ultimate concrete compressive strains (and the corresponding ultimate curvatures) correspond to rupture of the jackets of the retrofit specimens, while most of the failures of the retrofit specimens involved buckling and rupture of the longitudinal reinforcing bars.

(12) The experimentally determined displacement ductility capacity of the retrofit specimens was significantly greater than the displacement ductility capacity of the non-retrofit specimens. The displacement ductility capacity of the retrofit specimens increased with the strength and stiffness (i.e., the number of plies) of the jackets. A jacket with as few as two plies of carbon fiber tow sheet significantly increased the ductility capacity. The experimentally determined displacement ductility capacity of the retrofit specimens was greater than expected from the analytical results.

(13) The jacket strain increases with displacement ductility level and with cyclic loading at a single ductility level. Eventually, the jacket begins to bulge, and this bulging was observed visually and also appears as an increase in jacket strain where the bulging occurs.
7.3 CONCLUSIONS

The following conclusions are drawn from the investigation:

(1) Retrofit of non-ductile reinforced concrete building columns with carbon FRP jackets can significantly increase the ductility capacity of these columns. Depending on the existing strength of the building structure, this increased ductility capacity may enable the structure to survive moderate to severe seismic loading.

(2) The shear strength of a non-retrofit ductile shear building column can be predicted by adding the shear resistance of the transverse reinforcing steel, \( V_s' \), to the shear resistance of the concrete, \( V_c' \), which degrades with increasing flexural ductility levels (Priestley et al. 1994). The shear strength the retrofit ductile shear specimens can be increased sufficiently to alter the failure mode to the axial flexural mode.

(3) The limit states of non-ductile reinforced concrete building columns retrofit with carbon FRP jackets are: jacket rupture and longitudinal reinforcing bar rupture. With a well-designed jacket, the ultimate limit state of the retrofit column is rupture of the longitudinal reinforcing bar. This limit state occurs as a result of loss of confinement of the longitudinal reinforcing bar because of damage to the concrete underneath the jacket, allowing the longitudinal reinforcing bar to buckle which leads to rupture. The limit state of jacket rupture occurs as a result of insufficient jacket strength and stiffness.

(4) FRP jacket design guidelines for retrofit of bridge pier columns proposed by Seible et al. (1995) provide a conservative retrofit design for reinforced concrete building columns similar to those considered in this investigation. Jackets designed with substantially less material performed nearly as well.

(5) Fiber cross-section analyses provide an accurate predication of the concrete compressive
strain and curvature of a retrofit non-ductile reinforced column at failure. However, the
displacement ductility at failure may be greater than expected due to contributions to
displacement from pullout of longitudinal reinforcement at the base of the column and
shear deformations.

7.3 FUTURE RESEARCH

This investigation was not intended to provide conclusive findings regarding the design
and behavior of non-ductile reinforced concrete building columns retrofit with FRP jackets. Non­
ductile building columns include a wide range of possible dimensions, details, and non-ductile
behavior. FRP jackets can be made from a variety of materials including carbon, glass, and
aramid fibers. This study considered a limited number of test specimens with similar dimensions,
details, and behavior and considered FRP jackets made from carbon fiber tow sheet. As noted in
Chapter 1, work on non-ductile building columns exhibiting lap splice failure and brittle shear
failure modes is ongoing. In addition, more research related to the axial flexural failure and
ductile shear failure mode are needed.

A more comprehensive investigation of the relationship between FRP jacket stiffness and
strength, the extreme fiber concrete compressive strain, and the limit states of jacket rupture
and/or longitudinal reinforcing bar rupture is needed. Full-scale tests should be used to develop
relationships which will allow the extreme fiber concrete compressive strain to be used to reliably
predict jacket rupture and/or longitudinal reinforcing bar rupture as a function of the FRP jacket
properties.

Based on these relationships, more reliable analysis methods can be developed to predict
the ductility capacity of non-ductile building columns retrofit with FRP jackets against the axial
flexural and ductile shear failure modes. These analysis methods can be incorporated into design

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guidelines for FRP jackets. Using these guidelines, a number of FRP jacket retrofits for columns should be designed, and the prediction of ductility capacity verified through full-scale testing.
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VITA

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