Seismic Behaviour of Reinforced Concrete Columns

by

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Abstract

Appropriate transverse confinement can significantly improve strength, ductility and energy dissipation capacity of reinforced concrete columns, therefore enhancing their seismic resistance. This study is conducted to evaluate the seismic behaviour of concrete columns transversely confined by steel spirals, ties or fiber reinforced polymer (FRP) wrapping.

In the experimental program of this study, fifteen circular concrete columns of 356 mm (14 in.) diameter and 1473 mm (58 in.) length were tested under lateral cyclic displacement excursions while simultaneously subjected to constant axial load thus simulating earthquake loads. Eight columns were solely confined by various amounts of steel spirals, while seven other columns containing only minimal steel spirals were retrofitted by external FRP wrapping. Test results revealed that the increased transverse confinement can improve the energy dissipation capacity, ductility, deformability and flexural strength of concrete columns. The required transverse confinement should also be enhanced with the increase of axial load level to satisfy certain seismic design criterion.

A computation program was developed to conduct monotonic pushover analysis for confined concrete columns, which can predict the envelope curves of moment vs. curvature and shear vs. deflection hysteresis loops with reasonable accuracy for columns subjected to simulated seismic
loading. Based on extensive numerical analysis, expressions were developed for the relationships between the amount of transverse confinement and different ductility parameters, as well as the strength enhancement of confined columns. Finally, design procedures to determine the amount of transverse confinement were developed for concrete columns to achieve a certain ductility target. The enhancement of flexural strength of columns due to transverse confinement was also evaluated.
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Notation

\(a\) = parameters in the volumetric strain model (Chapter 2)
\(=\) effective factor of transverse confinement (Chapter 2)
\(=\) a dimension in column testing setup (Chapter 4)
\(A\) = cross-sectional area of column
\(A_b\) = cross-sectional area of a longitudinal steel bar
\(A_c\) = area of core concrete measured to the centerline of peripheral transverse reinforcement
\(A_{ch}\) = cross-sectional area of concrete core, measured to the outer surface of peripheral transverse reinforcement
\(A_{\text{eff}}(x)\) = effective shear area of column
\(A_g\) = gross cross-sectional area of column
\(A_s\) = cross-sectional area of a reinforcing steel bar
\(A_{sh}, A_{shy}\) = total effective cross-sectional area of transverse reinforcing steels in the direction under consideration
\(A_{sh\ (ACI)}\) = total effective cross-sectional area of transverse reinforcing steels required by ACI 318
\(A_{st}\) = total area of longitudinal reinforcement
\(A_{te}\) = area of one leg of transverse reinforcement
\(b\) = parameters in the proposed volumetric strain model
\(=\) side length of rectangular cross section
\(=\) a dimension in column testing setup (Chapter 4)
\(b_c, d_c\) = core sizes to centerline of peripheral transverse reinforcement in two main directions of rectangular cross-sections, respectively
\(b_o\) = width of cross section of confined concrete core inside centerline of peripheral transverse reinforcement
\(B\) = side size of square columns reported by Bae (2005)
\(c\) = parameters in the volumetric strain model (Chapter 2)
\(=\) a dimension in column testing setup (Chapter 4)
\[ c = \text{distance from extreme compression fiber to neutral axis at column-footing interface (Chapter 5)} \]
\[ = \text{a parameter in constitutive relationship of concrete (Chapter 5)} \]
\[ cc = \text{thickness of clear concrete cover} \]
\[ c_y = \text{width of concrete core in columns, normal to the direction under consideration and measured to the centerline of peripheral ties} \]
\[ d = \text{distance from extreme compression fiber of concrete to centroid of extreme layer of longitudinal tensile steel} \]
\[ d_b = \text{diameter of longitudinal steel reinforcement} \]
\[ d_s = \text{core size to centerline of circular spiral or peripheral hoop normal to the direction under consideration} \]
\[ D = \text{diameter of circular columns or depth of square columns} \]
\[ D_{md} = \text{distance from column-stub interface to the critical section} \]
\[ D_{dr} = \text{distance from stub face to right end of the most damaged region} \]
\[ E_c = \text{secant modulus measured at a stress of } f_{pl} \]
\[ E_{c,\text{eff}} = \text{effective secant modulus of elasticity of concrete, assessed from the compressive stress and strain at extreme compression fiber of concrete core} \]
\[ E_F = \text{Young's modulus of FRP jackets} \]
\[ E_i = \text{secant modulus corresponding to } (\varepsilon_i, f_i), E_i = f_i / \varepsilon_i \]
\[ E_s = \text{elastic modulus of steel reinforcement} \]
\[ E_{ti} = \text{initial tangent modulus at zero stress} \]
\[ E_{2i} = \text{secant modulus corresponding to } (\varepsilon_{2i}, f_{2i}), E_{2i} = f_{2i} / \varepsilon_{2i} \]
\[ f_c = \text{compressive strength of unconfined concrete} \]
\[ f'_c = \text{compressive strength of unconfined concrete} \]
\[ f^*_c = \text{compressive strength of confined concrete} \]
\[ f_{cc}' = \text{peak stress in stress-strain curve of confined concrete} \]
\[ f_F = \text{stress of in FRP jackets} \]
\[ f_{Fj} = \text{workable tensile strength of FRP jackets in the hoop direction of columns} \]
\[ f_{Fu} = \text{ultimate tensile strength of FRP jackets} \]
\[ f_i = \text{concrete stress corresponding to } \varepsilon_i \]
\[ f_i = \text{effective transverse stress} \]
$f_{i,a}$ = actual maximum confining pressure at FRP rupture

$f_{i,n}$ = nominal confining stress in FRP-confined columns, determined based on the tensile strength, $f_{Ru}$, of FRP jackets from the FRP coupon tests

$f_{ie}$ = effective confining pressure in concrete

$f_{pl}$ = 0.45 $f'_c$

$f_u$ = ultimate strength of reinforcing steel

$f_y$ = yield strength of reinforcing steel

$f_{yh}, f_{yw}$ = yield strength of transverse reinforcing steel

$f_{2i}$ = concrete stress corresponding to $\varepsilon_{2i}$

$G$ = shear modulus of concrete

$G_{eff}(x)$ = effective shear modulus of core concrete

$h$ = depth of cross section of a rectangular column

$h_c$ = external depth of cross section of the original unspalled column

$h_o$ = depth of cross section of confined concrete core inside centerline of transverse reinforcement

$H$ = height of a concrete column

$H_L$ = height of ribs on longitudinal steel bars

$I_e$ = confinement index

$I_e'$ = effective confinement index

$k_c$ = coefficient of confining effectiveness

$k_e$ = confinement effectiveness coefficient

$k_{\varepsilon_1}, k_{\varepsilon_2}$ = strain efficiency coefficient of FRP jackets

$k_m$ = factor considering the effect of FRP materials on the gradual rupture of FRP jackets

$k_m$ = factor accounting for the size effect on the gradual rupture of FRP jackets

$k_n$ = factor to allow for influence of steel configuration in rectangular columns

$k_p$ = axial load level, taken as $P/P_o$

$k_t$ = factor to account for the influence of mechanical ratio $\rho_m$ of longitudinal reinforcement on flexural strength enhancement of confined columns

$k_{\lambda F}$ = factor to account for the influence of shear span-depth ratio $\lambda$ on flexural strength enhancement of FRP-confined columns
$k_{s}$ = factor to account for the influence of shear span-depth ratio $\lambda$ on flexural strength enhancement of steel-confined columns

$k_1$ to $k_8$ = factors in numerical study (Chapter 6 and 7)

$K$ = factor in model by Alsiwat and Saatcioglu (1992)

$l, L$ = length of shear span of column

$l_p, L_p$ = length of equivalent plastic hinge

$L_d$ = maximum length of the elastic extension region in bar slip model

$L_{dr}$ = length of most damaged region in tested columns

$L_e$ = length of the elastic extension region in bar slip model when $\sigma_s \leq f_y$

$L_{sh}$ = length of plastic extension region in bar slip model when $\sigma_s > f_y$

$m$ = ratio of $f_y$ of longitudinal steel reinforcement to $0.85f'_c$ in NZS 3101:2006

$= \text{ratio of } f_y \text{ of longitudinal steel reinforcement to } f'_c \text{ of concrete}$

$M$ = moment in the section of a column

$M_{max}$ = maximum moment at the critical section

$M_n$ = nominal flexural strength

$n_F$ = number of layers of FRP jackets

$n_l$ = total number of longitudinal bars in column cross section which are transversely supported by the corners of ties or by standard hooks of cross ties in a rectangular column

$N_{Ed}$ = design axial force of column under load combination with earthquake effect

$p$ = confining pressure in concrete core

$P$ = axial load applied on columns

$P_f$ = factored axial load of column

$P_o$ = nominal axial load capacity of column as defined in ACI 318-11

$P_L$ = applied lateral force by the MTS actuator

$P_{ro}$ = factored axial load capacity of column

$q_o$ = basic behaviour factor in EN 1998-1:2004

$r_c$ = radius of rounded corners

$R_d$ = seismic force modification factor

$s, s_h$ = centre-to-centre spacing of adjacent transverse confining steels

= unzipping length of continuous FRP jackets, or clear spacing of discontinuous FRP-bands (Chapter 2)
\( s \) = clear spacing between adjacent transverse confining steels
\( S_L \) = clear spacing of ribs on longitudinal steel bars
\( t_F \) = thickness of FRP jackets
\( T_C \) = period at the upper limit of the constant acceleration region of spectrum
\( T_I \) = fundamental period of building within considered vertical plane of frame
\( u_e \) = average friction stress in the elastic extension region
\( u_f \) = average friction stress in plastic extension region
\( V \) = lateral shear load
\( V' \) = applied lateral force at the column tip
\( V_{\text{max}} \) = maximum shear force of column at failure
\( V_n \) = nominal lateral load capacity
\( w_i' \) = distance between adjacent longitudinal steel in rectangular cross-sections
\( x \) = distance of each section to the tip of column
\( Y_P \) = factor to reflect the influence of axial load of columns
\( \alpha \) = parameter in the model by Attard and Setunge (1996)
\( \alpha \) = factor to account the effect of different strain hardening character in model by Dhakall and Maekawa (2002)
\( \alpha_F \) = composite parameter in the numerical study of curvature ductility factor of FRP-confined concrete columns
\( \alpha_s \) = composite parameter in the numerical study of curvature ductility factor of steel-confined concrete columns
\( \beta \) = constant to account for different cross-sectional shapes and ductility levels
\( \beta_F \) = composite parameter in the numerical study of displacement ductility factor of FRP-confined concrete columns
\( \beta_s \) = composite parameter in the numerical study of displacement ductility factor of steel-confined concrete columns
\( \gamma_F \) = composite parameter in the numerical study of drift ratio and strength enhancement of FRP-confined concrete columns
\( \gamma_s \) = composite parameter in the numerical study of drift ratio and strength enhancement of steel-confined concrete columns
\( \delta \) = lateral drift ratio
\( \delta_L \) = deflection at the point of application of lateral load by the MTS actuator
\( \delta_{md} \) = lateral deflection at the critical section

\( \delta_{st} \) = slip extension of extreme tensile longitudinal steel of column out of footing

\( \Delta \) = lateral deflection

\( \Delta_{flexure} \) = tip deflection of column due to flexural deformation

\( \Delta_{max} \) = measured values of the maximum displacement

\( \Delta_{shear} \) = tip deflection of column due to shear deformation of the column

\( \Delta_{slip} \) = tip deflection of column due to fix end rotation of column

\( \Delta_{u} \) = ultimate deflection

\( \Delta_{y} \) = yield deflection

\( \varepsilon^{*} \) = strain at intermediate point

\( \varepsilon_{c} \) = strain of concrete corresponding to stress \( \sigma_{c} \)

\( \varepsilon_{c}' \) = strain corresponding to stress \( f_c' \) of unconfined concrete

\( \varepsilon_{c}^{lim} \) = \( \frac{1 - \nu_{s}}{\nu_{o}E_{c}} f_t - \frac{\varepsilon_{cr}}{\nu_{o}E_{c}} - \frac{2\nu_{o}f_t}{\nu_{o}E_{c}} \), axial strain corresponding to concrete cracking in transverse direction

\( \varepsilon_{c}^{vo} \) = axial strain when volumetric strain reaches zero

\( \varepsilon_{c,c}' \) = strain corresponding to peak stress \( f_{cc}' \) of confined concrete

\( \varepsilon_{cr} \) = strain corresponding to the splitting of concrete at stress \( f_{cr} = 0.387f_{c}^{0.63} \)

\( \varepsilon_{cu}, \varepsilon_{cu}^{*} \) = ultimate strain of confined concrete

\( \varepsilon_{F} \) = strain of FRP jackets

\( \varepsilon_{F,1} \) = maximum FRP strain in column test

\( \varepsilon_{F,2} \) = maximum FRP strain in coupon test

\( \varepsilon_{F,3} \) = ultimate strain of dry fabric reported by the manufacture

\( \varepsilon_{0,rup} \) = actual rupture strain of FRP jackets

\( \varepsilon_{i} \) = concrete strain at the point of inflexion on the descending branch of stress-strain curve in model by Attard and Setunge (1996)

\( \varepsilon_{m} \) = maximum recorded strains in spirals during testing

\( \varepsilon_{s} \) = strain in spirals

\( \varepsilon_{sh} \) = strain at the beginning of strain-hardening of reinforcing steel

\( \varepsilon_{su} \) = ultimate strain corresponding to the ultimate strength \( f_{u} \) of reinforcing steel
\[ \varepsilon_v = \varepsilon_c + 2\varepsilon_{1}, \text{volumetric strain of concrete} \]
\[ \varepsilon_u = \text{strain of reinforcing steel corresponding to } f_u \]
\[ \varepsilon_y = \text{yield strain of reinforcing steel} \]
\[ \varepsilon_{2i} = 2\varepsilon_i - \varepsilon_{cc}^{*}, \text{in model by Attard and Setunge (1996)} \]
\[ \xi_{cu}^{*} = \text{neutral axis depth of confined concrete core normalized to } h_0 \]
\[ \theta_{slip} = \text{angle of rigid rotation of column around the neutral axis} \]
\[ \lambda = \text{factor to account column size effect in model by Cui and Sheikh (2010)} \]
\[ = \text{empirical factor in EN 1998-1:2004} \]
\[ = \text{shear span-depth ratio of column} \]
\[ \mu_\phi = \text{curvature ductility factor} \]
\[ \mu_{\phi, in} = \text{increase in curvature ductility due to transverse FRP-confinement} \]
\[ \mu_{\phi, org} = \text{curvature ductility factor of deficient original columns before FRP-retrofit} \]
\[ \mu_\theta = \text{local ductility factor} \]
\[ \mu_\Delta = \text{displacement ductility factor of column} \]
\[ \mu_\delta = \text{global displacement ductility factor} \]
\[ \rho_{cc} = \text{area ratio of total longitudinal steel to concrete core} \]
\[ \rho_F = \text{volumetric ratio of FRP to confined concrete} \]
\[ \rho_{l, l} = \text{volumetric ratio of longitudinal reinforcement to gross cross section} \]
\[ \rho_s = \text{volumetric ratio of transverse reinforcement to gross cross section} \]
\[ = \text{volumetric ratio of transverse reinforcement to concrete core} \]
\[ \rho_s^{(ACI)} = \text{volumetric ratio of transverse reinforcement required by ACI 318-11} \]
\[ \rho_s^{(CSA)1} = \text{volumetric ratio of transverse reinforcement required by CAN/CSA-A23.3-04 for columns in ductile concrete frames} \]
\[ \rho_s^{(CSA)2} = \text{volumetric ratio of transverse reinforcement required by CAN/CSA-A23.3-04 for columns in moderately-ductile concrete frames} \]
\[ \rho_s^{(NZS)1} = \text{volumetric ratio of transverse reinforcement required by NZS 3101: 2006 for columns in ductile concrete frames} \]
\[ \rho_s^{(NZS)2} = \text{volumetric ratio of transverse reinforcement required by NZS 3101: 2006 for columns in moderately-ductile concrete frames} \]
\[ \rho_l = \text{area ratio of longitudinal reinforcement to gross cross section of column} \]
\( \rho_w \) = volumetric ratio of confining reinforcement to concrete core of column

\( \sigma^* \) = average stress corresponding to strain \( \varepsilon^* \) at intermediate point

\( \sigma_c \) = concrete stress

\( \sigma_l \) = point-wise stress corresponding to current strain \( \varepsilon_s \)

\( \sigma_l^* \) = point-wise stress corresponding to strain \( \varepsilon^* \) at intermediate point

\( \sigma_s \) = stress of steel corresponding to strain \( \varepsilon_s \)

\( \phi \) = resistance reduction factor in NZC-3101:2006

\( \phi_F \) = material strength reduction factor of FRP jackets

\( \phi_{\text{max}} \) = measured values of the maximum curvature

\( \phi_u \) = ultimate curvature

\( \phi_y \) = yield curvature

\( v \) = Poisson’s ratio of concrete

\( v_d \) = normalized design axial force of column

\( \omega_w \) = volumetric mechanical ratio of confining reinforcement

\( \Sigma A_b \) = sum of area of longitudinal reinforcement supported by tie under concern
Chapter 1
Introduction

1.1 General

Research on seismic resistance of structures has attracted increasing attention worldwide because of the economic and social implications of inadequate seismic design. Several researchers have reported (e.g. Yau and Sheikh, (1998)), that seismic upgrade or retrofit is urgently needed for over 200,000 bridges in North America due to seismic deficiency. More importantly, insufficient seismic resistance of columns is the most likely reason for the collapse of many concrete frame structures and bridges in major earthquakes, which has led to great loss of life in addition to economic loss. One hundred fifteen people lost their lives when the Canterbury Television building collapsed during the 2011 Christchurch earthquake in New Zealand due to insufficient seismic design and lack of structural ductility. About 70,000 people were killed in the 2008 Wenchuan Earthquake, China, with most of these fatalities caused by building collapse. Therefore, the importance of research on seismic upgrade of structures with the aim of developing innovative and feasible procedures cannot be overemphasized.

In most cases, it is not economically feasible to design concrete structures to resist severe earthquakes by elastic response and suffer only minimal damage, because of the low probability of occurrence of the very high level seismic event. The current methodology is to dissipate seismic energy through large inelastic deformation of structures. Many seismic provisions for concrete frames in modern design codes, such as ACI 318-11, CAN/CSA-A23.4-04, and NZC 3101:2006, are aimed at developing yielding in beams rather than in columns by adopting a “strong column-weak beam” philosophy. However, it has been proved that the commonly used strength ratio of 1.2 between column and beam cannot eliminate the occurrence of plastic hinges in frame columns during a major earthquake (Kuntz and Browning, 2003). One of the basic approaches to improve seismic resistance of concrete structures is to provide sufficient transverse confinement to the potential plastic hinge regions of columns to ensure appropriate ductile behaviour during severe earthquakes (Park and Paulay, 1975).
1.2 Problem

Even though extensive efforts have been made in research and engineering practice towards the safe design of concrete columns during the past several decades, the available information is still limited regarding their seismic behaviour when different materials, such as FRP, are used for transverse confinement. In particular, only a limited number of experiments have been conducted on realistically sized concrete columns transversely confined by steel reinforcement and/or FRP jackets under high axial load and lateral displacement excursions. Some analysis and design procedures were proposed based on individual experimental results, which cannot be validated due to the absence of a statistically relevant and comprehensive database of information. The seismic design provisions are quite different among different design codes for concrete columns. Therefore, a more focused experimental investigation and theoretical analyses are needed for the development of rational seismic design provisions.

1.3 Research objective

The present study is part of a comprehensive research program which has been underway at the University of Toronto for more than three decades (Sheikh, 1978; Sheikh and Uzumeri, 1980, 1982; Sheikh and Yeh, 1986; Sheikh and Khoury, 1993, 1997; Sheikh et al., 1994; Bayrak and Sheikh, 1998; Sheikh and Yau, 2002; Iacobucci et al., 2003; Memon and Sheikh, 2005; Ghosh and Sheikh, 2007; Sheikh and Li, 2007). The research objective is to investigate the seismic behaviour of concrete columns transversely confined by conventional steel and/or FRP reinforcement in their potential plastic hinge regions and to propose realistic design procedures for new structures as well as for the retrofit of existing ones.

In earlier stages of this research program, a large number of square columns with a range of concrete strengths and various types of confinement were tested. An analytical model for the mechanism of concrete confinement was proposed followed by the development of a performance-based design method for tied normal-strength concrete (NSC) square columns to achieve a given ductility performance (Sheikh and Uzumeri 1982; Sheikh and Khoury, 1997). This was extended later to high-strength concrete (HSC) and ultrahigh-strength concrete (UHSC) square columns (Bayrak and Sheikh, 1998). Sheikh and Li (2006) further developed this design procedure to the cases of FRP-confined concrete columns based on the then existing
limited test data. The experimental data, however, is still very limited especially for circular concrete columns transversely confined with steel spirals and/or FRP jackets and tested under realistic seismic loading. Availability of analytical procedures that can predict the behaviour of confined columns with reasonable accuracy is also limited.

In the present experimental research, fifteen circular concrete columns were tested under quasi-static lateral cyclic loading, while simultaneously subjected to constant axial compression simulating seismic loading. Each specimen consisted of a 356 mm (14 in.) diameter and 1473 mm (58 in.) long column, cast monolithically with a reinforced concrete stub of 510×762×813 mm (20×30×32 in.). Eight columns were confined solely by different amount of steel spirals, while seven other columns with only minimal spiral reinforcement were retrofitted by carbon fiber reinforced polymer (CFRP) or glass fiber reinforced polymer (GFRP) wrappings with fibers aligned in the circumferential direction of columns. The main variables of the tests are axial load level, spacing of spirals, and the amount and types of transverse FRP confinement.

A computer-aided analytical procedure was developed to conduct lateral pushover analysis for reinforced concrete columns and to predict the envelope curves of their hysteresis loops of moment vs. curvature and shear vs. tip deflection relationships when subjected to seismic loading. This procedure corroborated against realistic experimental data would be able to cover more complex conditions encountered in engineering practice and help develop a widely applicable design procedure. Based on extensive numerical analysis, expressions were developed to predict different ductility parameters and strength enhancement of confined columns. Finally, procedures were developed for the design of confinement reinforcement to meet a given ductility demand of columns and to estimate the strength enhancement caused by confinement. It should be noted that in the capacity design of columns, the underestimation of flexural strength is not always conservative.

Overall, the research presented in this thesis consists of both experimental and analytical studies of a broad range of columns types, which include rectangular and circular cross sections, and both steel-confined and FRP-retrofitted concrete columns. All these studies are within a consistent framework with the aim of providing useful recommendations for the structural engineering profession.
1.4 Thesis Organization

Chapter 2 presents a literature review of relevant work in this area. First, the main research about constitutive relationship of concrete confined by transverse steel or FRP jackets are introduced. Then, several important experimental studies on the seismic behaviour of confined concrete columns are reviewed. The code provisions and performance-based seismic design procedures for steel- or FRP-confinement to concrete columns are also reviewed and compared at the end of this chapter.

Chapter 3 introduces the newly conducted experiments on fifteen circular columns. The properties of materials, configuration of specimens, construction process, instrumentation, test setup, and testing procedures are all described in detail.

The experimental results and data analysis are presented in Chapter 4. Following the reporting of experimental observations, the measured test data is presented as diagrams and charts of shear vs. tip deflection and moment vs. curvature responses of columns. The effects of different variables on seismic behaviour of columns and the important observations are discussed in detail.

In Chapter 5 the development of a computational program is introduced, which includes the constitutive relationship of each material, procedures of sectional analysis and pushover analysis. The program is verified using the experimental results from large-scale columns.

Chapter 6 shows the results of a numerical parametric analysis for the seismic behaviour of confined columns. The developing trends of curvature ductility factor, displacement ductility factor and lateral drift ratio are studied and modelled. Also, the strength enhancement of columns due to transverse confinement is also evaluated.

Based on the analytical results from Chapter 6, seismic design procedures are developed in Chapter 7 to determine the demands on transverse confinement corresponding to various seismic criterions.

Finally, the conclusions and recommendations for further study are presented in Chapter 8.
Chapter 2
Literature Review

2.1 General

In this chapter, the previously conducted research on constitutive relationships of confined concrete, the seismic tests on concrete columns and current design provisions for transverse confinement are reviewed.

Extensive efforts have been made by researchers to develop the constitutive relationship of concrete transversely confined by steel reinforcement or FRP jackets as the fundamental requirement for evaluating the behaviour of confined concrete columns. A comprehensive discussion of the constitutive relationship of confined concrete is beyond this research program and can be found in Cui and Sheikh (2010b). In the following sections, the stress-strain relationship of confined concrete under axial compression and the methods to estimate the confinement efficiency in columns are briefly introduced. These are further discussed and adopted in Chapter 5 to develop a computation program for seismic analysis of confined columns.

Extensive research has been conducted on the seismic behaviour of concrete columns in the past several decades. The influences of factors, such as the transverse confinement and the axial load level, on the ductility performance of columns have been gradually revealed by the test data and analytical studies, which led to significant improvements in the design code provisions and performance-based seismic design procedures for confining reinforcement. The previously conducted seismic tests on steel-confined or FRP-confined concrete columns are reviewed in this chapter. Some selected experimental studies are discussed in detail, in which all specimens were tested under reversed cyclic lateral load while simultaneously subjected to constant axial load. Their performances were all reported in the forms of hysteresis loops of moment vs. curvature and lateral shear force vs. tip deflection curves and are used in this study to verify the analytical procedure in Chapter 5.

Lastly, seismic design provisions in different codes and proposed procedures to determine the required transverse confinement by steel or FRP jackets are discussed. It is found that the
confinement requirements among these provisions or methods varied dramatically which underlines the need for further research to achieve some consistent and reliable outcome in this realm.

2.2 Constitutive relationships of confined concrete

2.2.1 Concrete with steel-confinement

2.2.1.1 Introduction

The enhancement of compressive strength of concrete due to transverse confinement was originally reported by Considere (1903) and the first widely accepted relationship between strength enhancement and transverse confinement was proposed by Richart et al. (1928, 1929) for normal strength concrete confined by spirals or hydraulic pressure. Since then many experimental and theoretical investigations have been conducted on this research topic.

A general conceptual model for confinement by circular and rectilinear confining reinforcement was developed at the University of Toronto (Sheikh, 1978; Sheikh and Uzumeri, 1982) which formed the basis for a full stress-strain relationship for steel-confined concrete in tied columns under concentric compression. This was further extended to include the effect of strain gradient by Sheikh and Yeh (1986) for columns under seismic loading. The concept and determination of the effectively confined concrete area proposed by Sheikh and Uzumeri (1982) have been widely used by researchers for confined concrete columns since then. Mander et al. (1988) used this concept and developed a general stress-strain model applicable to normal-strength concrete columns with either circular or rectangular sections, under static and dynamic loading, and taking into account the effect of cyclic loading.

Among numerous analytical stress-strain models for concrete under constant transverse confinement, the stress-strain relationship proposed by Attard and Setunge (1996) has been verified by extensive test data to be the most successful in capturing the main characters for a wide range of concretes (from 36.4 MPa to 120 MPa) with different levels of confinement (from 5 MPa to 50 MPa) (Cui and Sheikh, 2010b). This model is introduced in the following section and also adopted later in the computation program presented in Chapter 5.
2.2.1.2 Attard and Setunge (1996)

This constitutive relationship of constantly-confined concrete (referred to as Attard and Setunge Model) consists of a unified equation. It is introduced below, with different sets of constants $A_s$, $B_s$, $C_s$, and $D_s$ for the pre-peak ascending branch and the post-peak descending branch of stress-strain ($\sigma_c$-$\varepsilon_c$) relationship, respectively.

\[ Y = \frac{A_s X + B_s X^2}{1 + C_s X + D_s X^2} \]  
\[ X = \frac{\varepsilon_c}{\varepsilon_{cc}}, \quad Y = \frac{\sigma_c}{f_c'} \quad (0 \leq Y \leq 1) \]  

1) For the ascending branch of unconfined concrete, the four constants are determined by:

\[ A_s = \frac{E_t \varepsilon_{cc}'}{f_c'} \]  
\[ B_s = \frac{(A_s - 1)^2}{\alpha \left(1 - \frac{f_{pl}}{f_c'}\right)} + \frac{A_s^2 (1 - \alpha)}{\alpha^2 f_{pl} \left(1 - \frac{f_{pl}}{f_c'}\right)} - 1 \]  
\[ C_s = A_s - 2 \]  
\[ D_s = B_s + 1 \]

where

- $\sigma_c$ = concrete stress;
- $f_c'$ = compressive strength of unconfined concrete;
- $f_{cc}'$ = peak stress in stress-strain curve of confined concrete, and taken as $f_c'$ for unconfined concrete;
- $\varepsilon_c$ = strain of concrete corresponding to stress $\sigma_c$;
- $\varepsilon_{cc}'$ = strain corresponding to stress $f_c'$ of unconfined concrete;
- $\varepsilon_{cc}'$ = strain corresponding to peak stress $f_{cc}'$ of confined concrete, and taken as $\varepsilon_{cc}'$ for unconfined concrete;
- $\alpha = \frac{E_t}{E_c}$;
\( E_{ii} = \) initial tangent modulus at zero stress, estimated to be 1.17\( E_c \) and \( E_c \) for 20 MPa and 100 MPa concretes, respectively, and linearly interpolate in-between;

\( E_c = \) secant modulus measured at a stress of \( f_{pl} \);

\( f_{pl} = 0.45 f'_c \).

2) For the descending branch of unconfined concrete, the constant set \( A_s, B_s, C_s, \) and \( D_s \) are:

\[
A_s = \frac{f_{ic}(\varepsilon_i - \varepsilon_c)^2}{\varepsilon_c \varepsilon_{ic}(f'_c - f_i)}
\]

\[ B_s = 0 \]

\[ C_s = A_s - 2 \]

\[ D_s = 1 \]

where

\[
\frac{\varepsilon_{ic}}{\varepsilon_c} = 2.5 - 0.3 \cdot \ln(f'_c)
\]

\[
\frac{f_{ic}}{f'_c} = 1.41 - 0.17 \cdot \ln(f'_c)
\]

3) For the ascending branch of concrete transversely confined by constant pressure \( f_i \), the constants \( A_s, B_s, C_s, \) and \( D_s \) are evaluated by the same set of expressions as unconfined concrete. However, for descending branch of constantly confined concrete, the four constants should be reevaluated by the following equations:

\[
A_s = \left( \frac{\varepsilon_{2i} - \varepsilon_i}{\varepsilon_{ic}} \right) \left[ \frac{\varepsilon_{2i} E_i}{(f'_{cc} - f_i)} - \frac{4\varepsilon_i E_{2i}}{(f'_{cc} - f_{2i})} \right]
\]

\[
B_s = (\varepsilon_i - \varepsilon_{2i}) \left[ \frac{E_i}{(f'_{cc} - f_i)} - \frac{4E_{2i}}{(f'_{cc} - f_{2i})} \right]
\]

\[ C_s = A_s - 2 \]

\[ D_s = B_s + 1 \]
where

\[ E_i = \frac{f_i}{e_i}; \quad E_{2i} = \frac{f_{2i}}{e_{2i}}; \quad \text{and} \quad e_{2i} = 2e_i - e_{cc} \]  \hspace{1cm} (2-17)

\[ \frac{f_i}{f_{cc}} = \frac{0.41 - 0.17 \cdot \ln(f'_c)}{5.06 \cdot \left( \frac{f_i}{f'_c} \right)^{0.57} + 1} \]  \hspace{1cm} (2-18)

\[ \frac{\varepsilon_i}{\varepsilon_{cc}} = \frac{0.5 - 0.3 \cdot \ln(f'_c)}{1.12 \cdot \left( \frac{f_i}{f'_c} \right)^{0.26} + 1} \]  \hspace{1cm} (2-19)

\[ \frac{f_{2i}}{f_{cc}} = \frac{0.45 - 0.25 \cdot \ln(f'_c)}{6.35 \cdot \left( \frac{f_i}{f'_c} \right)^{0.62} + 1} \]  \hspace{1cm} (2-20)

4) It should also be mentioned that the peak stress \( f'_{cc} \) and corresponding strain \( \varepsilon'_{cc} \) were recalibrated with improved accuracy by Cui and Sheikh (2010b) based on extensive test data and are given below:

\[ \frac{f'_{cc}}{f_c} = \left( 1 + 10 \cdot \frac{f_i}{f_c} \right)^{0.6} \quad \text{(for} \quad f'_c < 60 \text{ MPa}) \]  \hspace{1cm} (2-21)

\[ \frac{f'_{cc}}{f_c} = \left( 1 + 14 \cdot \frac{f_i}{f_c} \right)^{0.5} \quad \text{(for} \quad f'_c \geq 60 \text{ MPa}) \]  \hspace{1cm} (2-22)

\[ \frac{\varepsilon'_{cc}}{\varepsilon_c} = 1 + \left[ 69.4 - 13.2 \cdot \ln(f'_c) \right] \left( \frac{f_i}{f'_c} \right) \]  \hspace{1cm} (2-23)

\[ \text{2.2.1.3 Confinement efficiency in steel-confined columns} \]

In steel-confined concrete columns, transverse confinement is usually provided by steel spirals, hoops or ties. The behaviour of confined concrete is affected by several variables including mechanical properties, amount and configuration of longitudinal and transverse reinforcement. Due to the discontinuity of steel reinforcement, not all the concrete core is effectively confined, as shown in Figure 2.1.
The following model for confinement efficiency was first developed by Sheikh and Uzumeri (1982) and further modified by Mander et al. (1988), in which the effective transverse confining stress $f_i$ in the concrete core is evaluated as:

$$
\begin{align*}
    f_i &= k_c \cdot \left( \frac{A_{sh} f_{yh}}{d_s s} \right) \\
    k_c &= \left( \frac{1 - \frac{s'}{2d_s}}{1 - \rho_{ec}} \right)^2 \\
    k_c &= \frac{1 - \frac{s'}{2d_s}}{1 - \rho_{ec}} \\
    k_c &= \frac{1 - \sum_{i=1}^{n} \left( \frac{w_i}{6b_c d_c} \right)^2 \left( 1 - \frac{s'}{2b_c} \right) \left( 1 - \frac{s'}{2d_c} \right)}{1 - \rho_{cc}}
\end{align*}
$$

(for columns with circular hoops) (2-25)

(for columns with circular spirals) (2-26)

(for rectangular sections) (2-27)
where: $k_c = \text{confinement efficiency coefficient}$; 

$A_{sh} = \text{total effective cross-sectional area of transverse reinforcing steels in the direction under consideration}$; 

$f_{yh} = \text{yield strength of transverse reinforcing steel}$; 

$d_s = \text{core size to centerline of circular spiral or peripheral hoop normal to the direction under consideration}$; 

$b_c, d_c = \text{core sizes to centerline of peripheral hoop in two main directions of rectangular cross-sections, respectively}$; 

$s' = \text{clear spacing between adjacent transverse confining steels}$; 

$s = \text{centre-to-centre spacing of adjacent transverse confining steels}$; 

$w_i = \text{distance between adjacent longitudinal steel in rectangular cross-sections}$; 

$\rho_{cc} = \text{area ratio of total longitudinal steel to concrete core}$.

### 2.2.2 Concrete with FRP-confinement

#### 2.2.2.1 Introduction

As an alternative to conventional confinement technologies, fiber-reinforced polymer (FRP) composites show great potential in replacing traditional steel reinforcement to retrofit concrete columns with deficient transverse reinforcement and have attracted considerable research in the past two decades. The material properties of FRP and various factors that affect FRP performance have been introduced in detail by Sheikh and Yau (2002) and Li (2003). The externally bonded FRP jackets with fibers aligned mainly in the circumferential direction can effectively provide confinement which leads to significant enhancement of compressive strength and deformability of concrete.

In steel-confined columns, the transverse steel may yield at the early stage of concrete deformation and the confining pressure keeps approximately constant afterwards. Therefore, the confining pressure is evaluated based on the yield strength of steel. On the contrary, FRP behaviour under tension is almost perfectly linearly elastic and the confining pressure applied by FRP wrapping does not remain constant with increased load. Due to this reason, the existing compressive stress-strain models for steel-confined concrete are not applicable for the concrete with transverse FRP-confinement (Mirmiran and Shahawy, 1997).
To accurately predict the seismic behaviour of FRP-confined concrete columns, a reliable stress-strain relationship of concrete is crucial. Based on a large database of test results, many design-oriented stress-strain models for FRP-confined concrete were empirically proposed (Lam and Teng, 2003, 2004). These models are simple to use while they reflected most of the main characteristics of concrete behaviour. However, the reliability of these models depends on the type of FRP composites and none of them can accurately capture the behaviour of FRP-confined high strength concrete. On the other hand, analytical constitutive models of FRP-confined concrete were developed by researchers such as Toutanji (1999) and Cui and Sheikh (2010b) based on incremental procedure which were more realistic and captured the mechanics of confinement. In the next section, an analytical constitutive model proposed by Cui and Sheikh (2010b) is introduced, since its feasibility and good accuracy have been verified by extensive test data with a wide range of concrete strength (from 20 MPa to 120 MPa) and various kinds of FRP materials. This model is adopted in Chapter 5 in the development of the computation program for seismic analysis of columns.

2.2.2.2 Cui and Sheikh (2010b)

An iterative incremental procedure was proposed by Cui and Sheikh (2010b) to determine the compressive stress-strain relationship of FRP-confined concrete. In this model, a stress-strain relationship of concrete under constant active confinement was used to evaluate the axial stress and strain of FRP-confined concrete under varying confining pressure. Interaction between concrete and FRP jackets was executed by using material properties, force equilibrium, and strain compatibility. The main components of Cui and Sheikh model are introduced here.

1) Iterative procedure

The flow chart is presented in the following figure for the iterative procedure used in Cui and Sheikh model to develop the stress-strain relationship of FRP-confined concrete. The full compressive stress-strain relationship is developed for FRP-confined concrete by increase the axial strain $\varepsilon_c$ from zero to the ultimate value.

For each applied axial strain $\varepsilon_c$, the corresponding axial stress $\sigma_c$ in concrete is evaluated by the stress-strain relationship of concrete under constant confinement with previously determined
transverse confining stress \( f_i \) in the last incremental step. Then, using a dilation model for confined concrete, the new value of transverse strain is calculated by \( \varepsilon_l = \frac{1}{2} (\varepsilon_v + \varepsilon_c) \) for this incremental step, with \( \varepsilon_c \) (volumetric strain) and \( \varepsilon_l \) taken as positive when dilating. This updated value of \( \varepsilon_l \) can then be used for a new estimate of \( f_i \), leading to an iterative procedure until \( f_i \) converges to a stable value. The whole procedure is repeated for each incremental \( \varepsilon_c \) over the complete stress-strain curve until the final failure of the FRP confinement.

![Flow chart](image)

Figure 2.2 Flow chart for developing stress-strain relationship of FRP-confined concrete

2) Constitutive model of concrete under constant confinement

The previously introduced stress-strain relationship proposed by Attard and Setunge (1996) was used in the Cui and Sheikh model for concrete under certain level of transverse confinement by FRP jackets, due to its wide range of applicability.

3) Dilation model

The dilation characteristics of confined concrete are represented on the pattern of the volumetric strain model proposed by Imran and Pantazopoulou (1996) as follows:
Before cracking of confined concrete in transverse direction the volumetric strain is:

$$
\varepsilon_v = (1 - 2\nu_o) \cdot \left( \frac{2f_l}{E_c} + \varepsilon_c \right) \quad \text{(for } \varepsilon_c \leq \varepsilon_c^{lim}) \quad (2-28)
$$

After cracking:

$$
\varepsilon_v = (1 - 2\nu_o) \cdot \left[ \frac{2f_l}{E_c} + \varepsilon_c^{vo} \cdot \left( \frac{\varepsilon_c}{\varepsilon_c^{vo}} - b \cdot \left( \frac{\varepsilon_c - \varepsilon_c^{lim}}{\varepsilon_c^{vo} - \varepsilon_c^{lim}} \right) \right) \right] \quad \text{(for } \varepsilon_c > \varepsilon_c^{lim}) \quad (2-29)
$$

where

- $\varepsilon_v = \varepsilon_c + 2\varepsilon_l$, volumetric strain of concrete;
- $\varepsilon_c^{lim} = \frac{1 - \nu_o}{\nu_o E_c} f_l - \varepsilon_{cr} - \frac{2\nu_o f_l}{E_c}$, axial strain corresponding to concrete cracking in transverse direction;
- $\varepsilon_{cr} = \text{strain corresponding to the splitting of concrete at stress } f_{cr} = 0.387f_c^{0.63};$
- $\varepsilon_c^{vo} = \text{axial strain when volumetric strain reaches zero, taken as } a\varepsilon_{cc}';$
- $f_l = \text{effective transverse stress}.$

and the McCauley brackets $\langle \rangle$ are defined as $\langle x \rangle = 0.5[x + \text{abs}(x)].$

The parameters $a, b$ and $c$ in the above expressions were proposed by Cui and Sheikh (2010b) as:

$$
0.65 \leq a = \frac{f_c'}{\{f_c' - 50\} + 40} - 0.1 \leq 1.1 \quad (2-30)
$$

$$
b = 1 - \frac{f_l}{f_c'} \geq 0.7 \quad (2-31)
$$

$$
c = \frac{f_c' - f_l}{30} \geq 2.0 \quad (2-32)
$$

4) **FRP confinement stress**

The stress-strain relationship of FRP jackets is assumed to be perfectly linear elastic until rupture. For circular cross section of diameter $D$, the transverse confining stress is

$$
f_l = \frac{1}{2} \rho_F \cdot f_F = \frac{2t_F}{D} \cdot E_F \cdot \varepsilon_F \quad (2-33)
$$
where

\[ \rho_F = \text{volumetric ratio of FRP to confined concrete}; \]
\[ f_F = \text{stress of in FRP jackets}; \]
\[ t_F = \text{thickness of FRP jackets}; \]
\[ E_F = \text{Young's modulus of FRP jackets}; \]
\[ \varepsilon_F = \text{strain of FRP jackets}; \]
\[ D = \text{diameter of concrete columns}. \]

The evaluation of transverse confining stress in FRP-confined rectangular columns will be introduced in the later section.

5) Effective strength of FRP-confinement

Based on the measured rupture strain of FRP jackets in concentric compression tests of small-scale FRP-confined concrete cylinders, some researchers have reported that the workable rupture strain in FRP-confined columns was significantly lower than the ultimate strain \( \varepsilon_{Fu} \) measured in the FRP coupon tension tests. Lam and Teng (2004) suggested the effectiveness coefficient of strength be 0.637 for CFRP jackets and 0.744 for GFRP jackets. However, the observation in the seismic tests of FRP-confined columns showed that, even after the first occurrence of FRP rupture, the large-scale columns could still undergo several cycles of lateral excursions. This phenomenon indicated that a gradual failure of FRP jackets exists in large-scale columns (Sheikh, et al., 2007). Cui and Sheikh (2010b) explained this phenomenon by an FRP-unzipping model and proposed the following procedure to predict the effective strength of FRP-confinement.

For circular columns, the strength effectiveness coefficient of continuous FRP-jackets is

\[ k_e = 1 - \frac{s}{2D} \tag{2-34} \]

where \( s = \text{unzipping length of FRP jackets}, \) and \( D = \text{diameter of concrete columns}. \) This expression can also be used to estimate the effective confinement in columns wrapped by discontinuous FRP-bands by taking \( s = \text{clear spacing between adjacent FRP-bands}. \) For columns with rectangular cross sections, the geometric efficiency should further be taken into account in the assessment of \( k_e, \) which will be described in detail in the next section.
The value of $s$ is obtained from the following expression during the unzipping procedure of the continuous FRP jacket, corresponding to the tensile strain $\varepsilon_F$ in FRP jackets:

$$\frac{s}{2D} = k_s \cdot \frac{\varepsilon_F - \text{SEF} \cdot \varepsilon_{Fu}}{[\varepsilon_{Fu}(1 - \text{SEF})]^{k_m}} \leq 1.0$$  \hspace{1cm} (2-35)

where $k_m$ = factor considering the effect of FRP materials on the gradual rupture of FRP jackets, calibrated as 0.68 based on test data; $k_s$ = factor accounting for the size effect on the gradual rupture of FRP jackets, taken as

$$k_s = \left( \frac{500}{D} \right)^2 \geq 2 \hspace{1cm} (D \text{ is in unit of mm})$$  \hspace{1cm} (2-36)

Gradual rupture of FRP jackets is assumed to start at a strain of $\varepsilon = \text{SEF} \cdot \varepsilon_{Fu}$. Before this strain, $s$ is taken as zero. After that, $s$ increases as the column is further strained until $s=2D$ or the tensile strain in FRP jackets reaches the ultimate strain $\varepsilon_{Fu}$, whichever occurs first.

The above mentioned strain efficiency factor ($\text{SEF}$) is estimated as:

$$\text{SEF} = \lambda \cdot \frac{600}{f'_{c} \cdot \lambda f_{Fu}} \leq \lambda$$  \hspace{1cm} (2-37)

where, $f_{Fu} = E_F \cdot \varepsilon_{Fu}$, is the ultimate strength of FRP measured in FRP coupon tension tests, and the factor $\lambda$ is to account the effect of column size, established as

- $\lambda = 0.6 \; \text{for} \; D \geq 300 \; \text{mm}$;
- $\lambda = 1.0 \; \text{for} \; D \leq 200 \; \text{mm}$;

and with linear interpolation for in-between values.

### 2.2.2.3 Confinement efficiency of FRP-confined rectangular columns

Since confining pressure by FRP jackets is not evenly distributed in the FRP-confined columns with rectangular cross sections, the following approach recommended by ACI 440.2R-08 is used conservatively to estimate the geometric efficiency in this situation.

For a rectangular cross section, transverse confining stress $f_l$ is taken as the corresponding value of an equivalent circular cross section with a diameter $D$ equal to the diagonal of the cross
section, as shown in Figure 2.3.

\[ D = \sqrt{b^2 + h^2} \]  

(2-38)

Figure 2.3 Equivalent circular cross section (ACI 440.2R-08)

Thus, the confinement efficiency coefficient \( k_e \) in a rectangular cross section is further developed from Eq. (2-34) as

\[
k_e = \left(1 - \frac{s}{2D}\right) \cdot \frac{A_e}{A_c}
\]

(2-39)

\[
A_e - A_c = \frac{3A_g}{1 - \rho_g} - \rho_g
\]

(2-40)

where:

- \( A_e \) = cross-sectional area of effectively concrete section;
- \( A_c \) = cross-sectional area of concrete;
- \( A_g \) = gross area of column cross section;
- \( b \) = side length of rectangular cross section;
- \( h \) = side depth of rectangular cross section;
- \( r_c \) = radius of rounded corners;
- \( \rho_g \) = volumetric ratio of longitudinal reinforcement to gross cross section;
- \( s \) = unzipping length of continuous FRP jackets, or clear spacing of discontinuous FRP-bands, same as defined in Eq.(2-34) in the preceding section.
2.3 Seismic tests on confined concrete columns

2.3.1 Columns with steel-confinement

2.3.1.1 Introduction

The experimental studies by Sheikh and Khoury (1993), Sheikh et al. (1994), Bayrak and Sheikh (1998), Bae (2005), Hossien et al. (2005) and Paultre et al. (2009) are summarized in the following sections. In these experiments, the steel-confined circular and square columns were tested under reversed cyclic flexure while simultaneously subjected to constant axial load with concrete strength ranging from 30 MPa to 120 MPa. The experimental results were reported in the form of hysteresis loops of moment vs. curvature and lateral shear force vs. tip deflection relationships for each specimen.

2.3.1.2 Sheikh and Khoury (1993) and Sheikh, Shah and Khoury (1994)

As parts of the comprehensive experimental program, tests on nine concrete columns were reported in these two papers. The concrete strength was around 30 MPa for six columns tested by Sheikh and Khoury (1993) and 55 MPa for the other three by Sheikh et al. (1994). Each specimen consisted of a 1473 mm long column with a 305×305 mm square cross section and connected to a 508×762×813 mm massive reinforced concrete stub. Each specimen represented a column or a structural member between the point of maximum moment and contraflexure point with the stub standing for a footing or a beam-column joint. The geometry of specimens, transverse steel configurations and test setup are shown in Figure 2.4 and Figure 2.5. It is found that the shear span of each column was 1841 mm, measured from the contraflexure point (i.e. centre of the right hinge) to the stub-column interface, which led to a shear span-depth ratio of about 6.0.

Each column contained eight US#6 longitudinal steel reinforcement, uniformly distributed around the column core, which had the dimension of 267×267 mm measured to the centrelines of peripheral ties, giving a core area equal to 77% of the gross cross section of column. The variables in these tests were the amount of transverse confined steel, steel configuration, and level of axial load. The specimen details are listed in Table 2.1 and Table 2.2.
The following observations and conclusions were reported from these tests:

1) The measured ductility and energy dissipation capacity of the columns increase dramatically with the enhancement of confinement. Furthermore, ties with configurations $A$ and $F$ could provide more efficient confinement than similar amount of ties with configuration $E$. The specimens with ties of configurations $A$ and $F$ could undergo more than twice the number of cycles and almost twice the lateral deflection compared with the specimens with ties of
configuration $E$. It was also noticeable that the closely spaced inner and peripheral ties yielded, but did not enter the strain-hardening region until the last several cycles before failure. While configurations $A$ and $F$ behaved similarly in columns tested under low to moderate axial loads, the 90° hooks in configurations $F$ opened up at large deformations in columns with high axial loads. The use of 90° hook was thus recommended for low to moderate axial loads only.

2) The required amount of transverse confinement should be significantly increased with the increase of axial load level to achieve certain curvature ductility target. It was found that, if the applied axial load was measured in terms of $P/P_o$ rather than $P/A_{gf_c}$, the required amount of transverse confinement was proportional to concrete strength $f_c$ for the columns with similar ductility performance and under similar axial load level.

3) At the commencement of cover concrete crushing, the maximum compressive strain at the extreme concrete fiber was about 0.0045 in normal strength concrete columns and 0.002 to 0.0035 in high strength concrete columns. The first visible crushing of cover concrete occurred during the third or fourth cycle of lateral loading in majority specimens when the lateral deflection was increased to about twice the nominal yield displacement $\Delta_y$.

4) Extensive buckling of longitudinal steel reinforcement occurred during the last cycle of lateral loading in most cases and indicated the commencement of failure.

5) The restraint provided by the stubs strengthened the adjacent critical sections and pushed the failure away from the stubs. In all the specimens, the failure initiated at a distance ranging from 225 to 325 mm away from the stub surface and later extended toward the stubs. The region over which this effect was significant was approximately equal to the column section dimension. It was recommended that, for safe estimate of the design shear force based on the flexural capacities of columns, appropriately reduced length of shear span should be used.

6) When columns were subjected to lower axial load and confined by the ties with steel configurations $A$ and $F$, the amount of confining steel required by the then ACI-318 code was found to be overly conservative to supply ductile performance. However, for columns under higher axial load and with comparable ductile performance, the code design was insufficient.
Table 2.1 Specimen details (Sheikh and Khoury, 1993, and Sheikh et al., 1994)

<table>
<thead>
<tr>
<th>Researcher</th>
<th>Specimen</th>
<th>Concrete strength, $f_c$ (MPa)</th>
<th>Tie type</th>
<th>Bar Size</th>
<th>$s$ (mm)</th>
<th>$\rho_s$ (%)</th>
<th>$f_{yh}$ (MPa)</th>
<th>$A_{sb}$</th>
<th>$A_{sb}(ACI)$</th>
<th>Axial load level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheikh and Khoury (1993)</td>
<td>FS-9</td>
<td>32.4</td>
<td>F</td>
<td>#3</td>
<td>95</td>
<td>1.68</td>
<td>507</td>
<td>1.46</td>
<td>2291</td>
<td>0.76</td>
</tr>
<tr>
<td></td>
<td>ES-13</td>
<td>32.5</td>
<td>E</td>
<td>#4</td>
<td>114</td>
<td>1.69</td>
<td>464</td>
<td>1.34</td>
<td>2298</td>
<td>0.76</td>
</tr>
<tr>
<td></td>
<td>AS-3</td>
<td>33.2</td>
<td>A</td>
<td>#3</td>
<td>108</td>
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<td>507</td>
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<td>507</td>
<td>1.52</td>
<td>2242</td>
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<td>A</td>
<td>#4</td>
<td>108</td>
<td>3.06</td>
<td>464</td>
<td>2.41</td>
<td>2349</td>
<td>0.77</td>
</tr>
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<td></td>
<td>AS-19</td>
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<td>A</td>
<td>#3&amp;6mm</td>
<td>108</td>
<td>1.30</td>
<td>507&amp;464</td>
<td>1.12</td>
<td>1412</td>
<td>0.47</td>
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<tr>
<td>Sheikh et al. (1994)</td>
<td>AS-3H</td>
<td>54.1</td>
<td>A</td>
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<td>#4</td>
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<td>4.30</td>
<td>464</td>
<td>2.01</td>
<td>3191</td>
<td>0.64</td>
</tr>
</tbody>
</table>

Table 2.2 Mechanical properties of steel reinforcement (Sheikh and Khoury, 1993, and Sheikh et al., 1994)

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Bar size</th>
<th>Diameter, $d_b$ (mm)</th>
<th>Area, $A_s$ (mm$^2$)</th>
<th>Yield strength, $f_y$ (MPa)</th>
<th>Elastic modulus, $E_s$ (MPa)</th>
<th>Yield strain, $\epsilon_y$</th>
<th>Begin of strain-hardening, $\epsilon_{sh}$</th>
<th>Ultimate strain, $\epsilon_u$</th>
<th>Ultimate strength, $f_u$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse</td>
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<td>71.3</td>
<td>507</td>
<td>187800</td>
<td>0.0027</td>
<td>0.0064</td>
<td>0.124</td>
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<tr>
<td></td>
<td>#4</td>
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<td>464</td>
<td>193300</td>
<td>0.0024</td>
<td>0.0118</td>
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<td>707</td>
</tr>
<tr>
<td>Longitudinal</td>
<td>#6</td>
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<td>181100</td>
<td>0.0028</td>
<td>0.0088</td>
<td>0.118</td>
<td>765</td>
</tr>
</tbody>
</table>

2.3.1.3 Bayrak and Sheikh (1998)

It is worth noting that only limited test data is available for large-scale high-strength concrete columns, tested under moderate to high levels of axial load and lateral cyclic displacement excursions, mostly due to the capacity limitation of experimental facilities. The results of an experimental research were reported by Bayrak and Sheikh (1998), which aimed to study the relationship between transverse steel confinement and the seismic behaviour of columns with high-strength concrete (HSC) and ultrahigh-strength concrete (UHSC).

Eight 305×305×1473 mm square columns were tested under lateral reversed cyclic displacement excursions while subjected to moderate to high levels of axial load. The concrete strength was about 72 MPa in four specimens and 102 MPa in the other four. Each column contained eight...
20M longitudinal bars, uniformly distributed around the column core, which had the dimension of 267×267 mm measured to the centreline of the perimeter ties (\(A_r/A_g=77\%\)). The geometry of specimens, test setup and procedure were similar to the previously introduced experiments by Sheikh and Khoury (1993) and Sheikh et al. (1994). The variables studied here were the concrete strength, axial load level, steel configuration, amount of transverse steel reinforcement, and the presence of a heavy stub. The specimen details and the mechanical properties of steel reinforcement are presented in Table 2.3 and Table 2.4.

Table 2.3 Specimen details (Bayrak and Sheikh, 1998)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Concrete strength, (f_c) (MPa)</th>
<th>Transverse reinforcement</th>
<th>Axial load level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tie type</td>
<td>Bar size</td>
<td>Spacing (mm)</td>
</tr>
<tr>
<td>ES-1HT</td>
<td>E</td>
<td>15M</td>
<td>95</td>
</tr>
<tr>
<td>AS-2HT</td>
<td>A</td>
<td>10M</td>
<td>90</td>
</tr>
<tr>
<td>AS-3HT</td>
<td>A</td>
<td>10M</td>
<td>90</td>
</tr>
<tr>
<td>AS-4HT</td>
<td>A</td>
<td>15M</td>
<td>100</td>
</tr>
<tr>
<td>AS-5HT</td>
<td>A</td>
<td>10M&amp;15M</td>
<td>90</td>
</tr>
<tr>
<td>AS-6HT</td>
<td>A</td>
<td>15M</td>
<td>76</td>
</tr>
<tr>
<td>AS-7HT</td>
<td>A</td>
<td>10M</td>
<td>94</td>
</tr>
<tr>
<td>ES-8HT</td>
<td>E</td>
<td>15M</td>
<td>70</td>
</tr>
</tbody>
</table>

Table 2.4 Mechanical properties of steel reinforcement (Bayrak and Sheikh, 1998)

<table>
<thead>
<tr>
<th>Reinforcement Bar size</th>
<th>Diameter, (d_b) (mm)</th>
<th>Area, (A_s) (mm²)</th>
<th>Yield strength, (f_y) (MPa)</th>
<th>Elastic modulus, (E_s) (MPa)</th>
<th>Yield strain, (\varepsilon_y)</th>
<th>Begin of strain-hardening, (\varepsilon_{sh})</th>
<th>Ultimate strain, (\varepsilon_u)</th>
<th>Ultimate strength, (f_u) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse</td>
<td>10M</td>
<td>11.28</td>
<td>100</td>
<td>542</td>
<td>210060</td>
<td>0.0026</td>
<td>0.0191</td>
<td>0.155</td>
</tr>
<tr>
<td></td>
<td>15M</td>
<td>15.96</td>
<td>200</td>
<td>463</td>
<td>193250</td>
<td>0.0024</td>
<td>0.0207</td>
<td>0.113</td>
</tr>
<tr>
<td>Longitudinal</td>
<td>20M</td>
<td>19.54</td>
<td>300</td>
<td>454</td>
<td>197560</td>
<td>0.0023</td>
<td>0.0067</td>
<td>0.129</td>
</tr>
</tbody>
</table>

The seismic behaviour of UHSC and HSC columns were examined and compared to normal strength concrete (NSC) columns. The following observations and conclusions were reported:

1) High strength concrete is much more brittle compared with normal strength concrete.

However experimental results showed that columns with 72 MPa and 102 MPa concrete could also perform in a ductile manner under high levels of axial load \((P/P_o=0.50)\) if
sufficient transverse confinement was provided. Thus, columns made of high strength concrete with appropriately designed transverse confining reinforcement can be confidently used in seismically active areas.

2) To achieve a certain level of curvature ductility factor, the required amount of transverse confinement is approximately proportional to the concrete strength for columns under similar axial load level measured as a fraction of the nominal concentric compression capacity, i.e. $P/P_o$.

3) An increase of axial load reduced the column’s deformability and ductility. Therefore, to achieve a certain level of ductile performance of columns, the required transverse confinement should be enhanced with the increase of axial load.

4) Based on the examination of the sectional and member ductility parameters of comparable specimens, it was found that the columns with higher strength concrete had lower deformability and energy absorption and dissipation capacities initially. However, during the larger displacement excursions, these properties improved rapidly and the total values were comparable to those columns with lower strength concrete.

5) The requirements in ACI 318-95 for the design of confinement reinforcement did not relate the amount of transverse steel reinforcement to the reinforcement configuration and the level of axial loads. It was suggested that these effects should be considered into the design of confining reinforcement.

2.3.1.4 Bae (2005)

In this experimental program four large-scale square columns with normal-strength concrete were tested under simulated seismic loading. All the specimens had the same geometric properties consisting of 610×610×2630 mm square columns connected with 965×965×2032 mm stubs, except that one specimen (S17-3UT) had a 438×438 mm square cross section. Twelve longitudinal steel bars were distributed uniformly around the concrete core in each column, leading to the same volumetric ratio of longitudinal reinforcement of 1.25%. The core size was 397×397 mm in the specimen S17-3UT and 523×523 mm in the other three columns, measured center-to-center of the peripheral hoops. The transverse reinforcement was arranged in
configurations as shown in the layout of specimens in Figure 2.6.

The column specimens were tested under quasi-static reversed cyclic lateral displacement excursions while subjected to constant axial loads. It should be noticed that the shear span of specimens was 3048 mm, measured from the column section with the maximum moment to the contra-flexural point of the column, which was the point of application of the lateral load in the test setup. Therefore the shear span-depth ratio of the column was 5.0 for the three S24 specimens and 7.0 for column S17-3UT.

The specimen details and mechanical properties of reinforcement are presented in the following Tables 2.5 and 2.6. Test results have been reported by Bae (2005) in the forms of hysteresis loops of moment vs. curvature and lateral force vs. tip deflection curves for each column.

![Figure 2.6 Layout of specimens (Bae, 2005)](image)

### Table 2.5 Specimen details (Bae, 2005)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Concrete strength, (f'_c) (MPa)</th>
<th>Size, (B) (mm)</th>
<th>Longitudinal reinforcement</th>
<th>Transverse reinforcement</th>
<th>Axial load level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Bar size</td>
<td>Ratio, (\rho_l) (%)</td>
<td>(f'_h) (MPa)</td>
<td>Bar size</td>
</tr>
<tr>
<td>S24-2UT</td>
<td>43.4</td>
<td>610</td>
<td>No.7-1</td>
<td>1.25</td>
<td>503</td>
</tr>
<tr>
<td>S17-3UT</td>
<td>43.4</td>
<td>438</td>
<td>No.5</td>
<td>1.25</td>
<td>496</td>
</tr>
<tr>
<td>S24-4UT</td>
<td>36.5</td>
<td>610</td>
<td>No.7-2</td>
<td>1.25</td>
<td>400</td>
</tr>
<tr>
<td>S24-5UT</td>
<td>41.4</td>
<td>610</td>
<td>No.7-2</td>
<td>1.25</td>
<td>400</td>
</tr>
</tbody>
</table>
The main objective of this research was to investigate the relationships between various criteria of seismic performance currently used by different performance-based confinement design procedures, which included curvature ductility factor, displacement ductility factor or lateral drift ratio of columns. The effects of shear span-depth ratio \((L/h)\), axial load level \((P/P_o)\), and the amount of confining reinforcement \((A_{sh})\) on the column behaviour were studied. The following comments can be made based on the test results:

1) The most important conclusion in this research is that the axial load level plays a very significant role in the ductility, strength, stiffness, and energy dissipation characteristics of the columns. The relationships between various ductility parameters are affected by the level of axial load. As the axial load level increases, the loss of lateral load carrying capacity due to the \(P-\Delta\) effect becomes higher. Therefore, the capacity of lateral drift ratio or displacement ductility of the column is limited under high levels of axial load, regardless of the amount of confinement. In other words, the loss of lateral load capacity due to the \(P-\Delta\) effect indicates that even though a column may have large curvature ductility, the maximum attainable lateral displacement of the column may be mainly controlled by the \(P-\Delta\) effects under high axial load.

2) The performance of test specimens can be evaluated differently depending on the ductility parameter used. Compared to column S24-4UT, the specimen S24-2UT had larger amount of confining reinforcement and was subjected to higher axial load. The reported test results showed that S24-2UT achieved a much lower sectional ductility capacity (lower curvature ductility factor), but a superior member-level ductility performance (higher displacement......
ductility factor) comparing to S24-4UT. This observation about the curvature ductility is in contrast to available information. Thus, if the displacement ductility factor is used as a criterion, it can be concluded that the detrimental effect of high axial load on S24-2UT were compensated sufficiently by its larger amount of confining reinforcement. However, if curvature ductility factor is used as the criterion, the opposite conclusion can be drawn.

3) Similar sectional performances do not produce similar member performances when shear span-to-depth ratios of concrete columns are different. The overall member performance of the test columns are significantly affected by the shear span-depth ratio. As the shear span-depth ratio decreases, the displacement ductility and drift capacity increase for a given curvature ductility level. For the same level of displacement ductility and drift capacity, different shear span-depth ratios would need different levels of sectional ductility.

4) For a given curvature ductility level, an increase in concrete strength results in the reduced displacement ductility and drift capacity. To achieve the target level of displacement ductility and drift capacity in a high strength concrete column, a larger amount of confining reinforcement should be provided.

5) It was found that a reduction of plastic hinge length from $h$ to $0.5h$ did not change the relationships between various ductility parameters significantly. Therefore, it can be concluded that the equivalent plastic hinge length is very important in predicting the individual column behaviour, especially for columns displaying limited ductility. However it did not affect the relationships of various ductility factors considerably in this study. This conclusion is contrary to the mechanics presented by Park & Pauley (1975), which related the displacement ductility factor to curvature ductility factor of cantilever columns under lateral seismic loading.

6) The author claimed that it was extremely difficult, if not impossible, to generalize the relationships accurately between various ductility parameters. The level of axial load, shear span-depth ratio, concrete strength, amount of longitudinal reinforcement and transverse reinforcement influence the deformation capacity of a column and the relationships between various ductility parameters.
2.3.1.5 Paultre, Eid, Robles, and Bouaanani (2009)

The test results of six large-scale circular columns with high-strength concrete (HSC) under simulated seismic loading were reported by Paultre et al. (2009). Each specimen consisted of a circular column with a diameter of 300 mm and a length of 2150 mm, connected with a massive I-shaped stub. The targeted concrete strength was 100 MPa. Six 20M Grade 400 deformed bars were provided in each column as longitudinal reinforcement, resulting in a volumetric ratio of 2.55%. The spiral reinforcement was provided with two types of steels: 10M Grade 400 and US#3 Grade 500 deformed reinforcement. The clear concrete cover was 20 mm, measured to the outer surface of spirals. The specimen details and mechanical properties of reinforcement are presented in the following Tables 2.7 and 2.8.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Concrete strength, ( f_c ) (MPa)</th>
<th>Bar Size</th>
<th>Spacing (mm)</th>
<th>( \rho_s ), (%)</th>
<th>( f_{sh} ) (MPa)</th>
<th>( \rho_s(ACI) )</th>
<th>( \rho_s(NZS) )</th>
<th>( \rho_s(CSA) )</th>
<th>Axial load level, ( P ) (kN)</th>
<th>( \frac{P}{f_c A_g} )</th>
<th>( \frac{P}{P_o} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>C100S100N15</td>
<td>109</td>
<td>10M</td>
<td>100</td>
<td>1.43</td>
<td>440</td>
<td>0.481</td>
<td>1.044</td>
<td>0.904</td>
<td>1156</td>
<td>0.15</td>
<td>0.16</td>
</tr>
<tr>
<td>C100SH100N15</td>
<td>100.5</td>
<td>#3</td>
<td>100</td>
<td>1.00</td>
<td>560</td>
<td>0.465</td>
<td>1.373</td>
<td>0.781</td>
<td>1066</td>
<td>0.15</td>
<td>0.16</td>
</tr>
<tr>
<td>C100S70N25</td>
<td>103</td>
<td>10M</td>
<td>70</td>
<td>2.04</td>
<td>440</td>
<td>0.728</td>
<td>0.784</td>
<td>0.817</td>
<td>1820</td>
<td>0.25</td>
<td>0.27</td>
</tr>
<tr>
<td>C100SH70N25</td>
<td>97</td>
<td>#3</td>
<td>70</td>
<td>1.43</td>
<td>560</td>
<td>0.689</td>
<td>0.857</td>
<td>0.698</td>
<td>1714</td>
<td>0.25</td>
<td>0.26</td>
</tr>
<tr>
<td>C100S37N40</td>
<td>100</td>
<td>10M</td>
<td>37</td>
<td>3.85</td>
<td>440</td>
<td>1.413</td>
<td>0.862</td>
<td>0.995</td>
<td>2827</td>
<td>0.40</td>
<td>0.43</td>
</tr>
<tr>
<td>C100SH37N40</td>
<td>103</td>
<td>#3</td>
<td>37</td>
<td>2.71</td>
<td>560</td>
<td>1.227</td>
<td>0.780</td>
<td>0.769</td>
<td>2912</td>
<td>0.40</td>
<td>0.43</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Bar size</th>
<th>Diameter, ( d_b ) (mm)</th>
<th>Area, ( A_s ) (mm²)</th>
<th>Yield strength, ( f_y ) (MPa)</th>
<th>Begin of strain-hardening, ( \varepsilon_{sh} )</th>
<th>Ultimate strain, ( \varepsilon_u )</th>
<th>Ultimate strength, ( f_u ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse</td>
<td>#3</td>
<td>9.53</td>
<td>71.3</td>
<td>560</td>
<td>-</td>
<td>0.050</td>
<td>697</td>
</tr>
<tr>
<td></td>
<td>10M</td>
<td>11.28</td>
<td>100</td>
<td>440</td>
<td>-</td>
<td>0.173</td>
<td>656</td>
</tr>
<tr>
<td>Longitudinal</td>
<td>20M</td>
<td>19.54</td>
<td>300</td>
<td>425</td>
<td>0.0084</td>
<td>0.112</td>
<td>679</td>
</tr>
</tbody>
</table>

The column specimens were tested under constant axial loads and reversed quasi-static lateral loads. The shear span of specimens was 2000 mm, measured from the base of the column with a maximum moment to the contra-flexural point of the column, which was the point of application.
of lateral load in the test setup. Thus the shear span-depth ratio of each column was 6.67. The experimental results were reported in the forms of hysteresis loops of moment vs. curvature and lateral force vs. tip deflection curves for each column.

The main focuses of this research were to study the post-elastic behaviour and the ductility factors of HSC circular columns designed according to the seismic design requirements for transverse steel in CAN/CSA-A23.3-04. The following conclusions were drawn by the authors based on the test results:

1) HSC circular columns subjected to different levels of axial load can achieve the same level of displacement ductility factor if the transverse reinforcement is designed properly which accounts for the axial load level and the yield strength of transverse steel.

2) Even though all the six specimens contained similar amount of confining reinforcement when non-dimensionalized with respect to the requirements in CAN/CSA-A23.3-04, the differences among the curvature ductility factors of six columns were significant, ranging from 11.4 to 26.0. However, the range of displacement ductility factors was not as large, varying between 4.8 and 7.4. The range of lateral drift ratios of the columns was also similar and varied from 6.1% to 8.2%.

3) The seismic design requirements in CAN/CSA-A23.3-04 and NZS 3101-95 of confining reinforcement showed good correlation with the displacement ductility of HSC columns. The test results indicated that the specimens designed according to these two standards could also display adequate sectional ductility. On the other hand, test results revealed that the requirements in the ACI 318 Code might be too conservative for columns under low level of axial load.

2.3.2 Columns with FRP-confinement

2.3.2.1 Introduction

Transverse FRP wrapping on concrete columns has become a popular alternative seismic retrofit technique in the last two decades due to its higher strength-weight ratio, easier operation and higher corrosion resistance compared to conventional steel and concrete jackets. However, the
experimental results available are still seriously limited for large-scale FRP-confined columns tested under seismic loading. Hence, the development of relevant design provisions is still lagging behind the practical application of FRP-retrofit to deficient columns.

Reviewed in the following sections are the experiments reported by Sheikh and Yau (2002), Gould and Harmon (2002), Iacobucci et al. (2003), Memon and Sheikh (2005), Hosseini et al. (2005), Ozbakkalogu and Saatcioglu (2006, 2007) and Wang et al. (2009), in which FRP-retrofitted concrete columns were tested under simulated earthquake effects which consisted of laterally reversed cyclic displacements and constant axial load applied simultaneously. The results for the seismic performance of columns were presented in the form of hysteresis loops of moment vs. curvature and lateral shear force vs. tip deflection relationships, except that the test results reported by Ozbakkalogu and Saatcioglu (2006, 2007) were in the form of hysteresis loops of moment vs. lateral tip deflection relationship.

2.3.2.2  Sheikh and Yau (2002)

Sheikh and Yau (2002) investigated the effectiveness of transverse FRP wrapping in strengthening deficient circular concrete columns or repairing damaged circular concrete columns. The seismic behaviour of substandard columns retrofitted by transverse FRP wrapping were compared with the conventional circular columns which were solely confined by steel spirals. The variables examined included: the amount and spacing of confining steels, the type and amount of FRP wraps, and the level of applied axial load. The specimen details and material properties are shown in Table 2.9 to Table 2.11.

In this experimental research, four columns were conventionally reinforced with longitudinal and spiral steel reinforcement, while the other five substandard columns were strengthened with continuous carbon fiber reinforced polymer (CFRP) or glass fiber reinforced polymer (GFRP). Each column had a diameter of 356 mm and a height of 1473 mm, and was attached at one end to a heavily reinforced 508×762×813 mm stub, which modeled a discontinuity such as a beam-column joint or a footing. Six 25M G400 steel deformed bars were used in all the columns as longitudinal reinforcement ($\rho_l = 3.0\%$), while US#3 G60 steel was used for spiral reinforcement at different spacing in each column. Clear concrete cover was 20 mm in all columns, measured to the outside surface of the spirals. Three types of continuous FRP wrapping (composed of
either carbon or glass fiber) were used to provide transverse external confinement to the five columns, with the unidirectional fiber aligned along the circumference of the columns.

### Table 2.9 Specimen details (Sheikh and Yau, 2002)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Concrete strength, $f_c$ (MPa)</th>
<th>Transverse reinforcement</th>
<th>Axial load level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Bar size</td>
<td>$s$, (mm)</td>
</tr>
<tr>
<td>S-1NT</td>
<td>40.1</td>
<td>US #3</td>
<td>80</td>
</tr>
<tr>
<td>S-2NT</td>
<td>40.1</td>
<td>US #3</td>
<td>80</td>
</tr>
<tr>
<td>S-3NT</td>
<td>39.2</td>
<td>US #3</td>
<td>300</td>
</tr>
<tr>
<td>S-4NT</td>
<td>39.2</td>
<td>US #3</td>
<td>300</td>
</tr>
<tr>
<td>ST-1NT</td>
<td>40.4</td>
<td>US #3</td>
<td>300</td>
</tr>
<tr>
<td>ST-2NT</td>
<td>40.4</td>
<td>US #3</td>
<td>300</td>
</tr>
<tr>
<td>ST-3NT</td>
<td>40.4</td>
<td>US #3</td>
<td>300</td>
</tr>
<tr>
<td>ST-4NT</td>
<td>44.8</td>
<td>US #3</td>
<td>300</td>
</tr>
<tr>
<td>ST-5NT</td>
<td>40.8</td>
<td>US #3</td>
<td>300</td>
</tr>
</tbody>
</table>

### Table 2.10 Mechanical properties of steel reinforcement (Sheikh and Yau, 2002)

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Bar size</th>
<th>Diameter, $d_b$ (mm)</th>
<th>Area, $A_s$ (mm$^2$)</th>
<th>Yield strength, $f_y$ (MPa)</th>
<th>Elastic modulus, $E_s$ (MPa)</th>
<th>Yield strain, $\varepsilon_y$</th>
<th>Begin of strain-hardening, $\varepsilon_{sh}$</th>
<th>Ultimate strain, $\varepsilon_u$</th>
<th>Ultimate strength, $f_u$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse</td>
<td>US#3</td>
<td>9.53</td>
<td>71.3</td>
<td>510</td>
<td>182143</td>
<td>0.0028</td>
<td>0.0095</td>
<td>0.130</td>
<td>785</td>
</tr>
<tr>
<td>Longitudinal</td>
<td>25M</td>
<td>25.2</td>
<td>500</td>
<td>490</td>
<td>196000</td>
<td>0.0025</td>
<td>0.0115</td>
<td>0.125</td>
<td>680</td>
</tr>
</tbody>
</table>

### Table 2.11 Mechanical properties of FRP composites (Sheikh and Yau, 2002)

<table>
<thead>
<tr>
<th>FRP type</th>
<th>Nominal thickness (mm/ply)</th>
<th>Tensile strength (MPa)</th>
<th>Rupture strain (mm/mm)</th>
<th>Elastic modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP</td>
<td>1.00</td>
<td>912</td>
<td>0.0142</td>
<td>64225</td>
</tr>
<tr>
<td>CFRP</td>
<td>0.50</td>
<td>1824</td>
<td>0.0142</td>
<td>128450</td>
</tr>
<tr>
<td>GFRP</td>
<td>1.25</td>
<td>414</td>
<td>0.0197</td>
<td>21015</td>
</tr>
</tbody>
</table>

All the columns were tested under simulated earthquake forces. The test setup and loading sequences of this experimental research were similar to those used by Sheikh and Khoury.
The 1841 mm long shear span of all the columns led to the shear span-depth ratio of 5.17. The hysteresis loops of moment vs. curvature and lateral shear force vs. tip deflection curves were reported by Sheikh and Yau (2002) for each column. The following conclusions were drawn based on the experimental results:

1) Substandard concrete columns can be effectively retrofitted by transverse FRP wrapping, which can significantly improve the seismic performance of columns and result in a remarkable increase of ductility level, energy dissipation capacity, as well as the flexural strength of columns.

2) Compared to the conventional steel spirals in concrete columns, FRP jackets provide a more effective confinement since the entire column section is wrapped inside the FRP jacket. The behaviour of the FRP-retrofitted columns under simulated seismic loading equalled or exceeded the performance of conventional steel-confined columns designed in accordance with the seismic provisions of ACI 318-95.

3) Columns retrofitted by FRP wrapping showed little degradation of flexural strength with the increase of lateral displacement excursion, which was distinguished from the seismic behaviour of conventional steel-confined columns.

4) In steel reinforced columns, section and member ductility decreased significantly as spiral spacing increased accompanied by a reduction in the transverse steel content. These adverse effects could be compensated by the confinement provided by the FRP jackets.

5) Column ductility degraded as the level of axial load increased. The amount of FRP reinforcement needed to retrofit substandard columns should be increased with the level of axial load in order to get satisfactory seismic behaviour.

2.3.2.3 Gould and Harmon (2002)

Twelve cantilever circular concrete columns, cast in GFRP tubes, were tested under constant axial load and lateral reversed cyclic shear. The columns had a diameter of 180 mm and a nominal length of 600 mm or 1200 mm. The test setup resulted in a shear span of approximately 530 mm or 1150 mm measured from the column base to the point of application of lateral load.
The exact shear span for each specimen is listed in Table 2.12. In each column, eight US#4 G60 deformed steels were evenly distributed around the circumference. The clear concrete cover was 10 mm measured to the outer surface of longitudinal steel and the volumetric ratio of longitudinal reinforcement $\rho_l = 4\%$. There was no transverse steel in any of the columns.

The prefabricated GFRP tubes with different ratios of Owens Corning 158 B-AC-250 E-Glass fibers were used as stay-in-place column formworks, as well as the transverse reinforcement. The unidirectional fibers in the GFRP tubes were aligned in the circumferential direction of columns. In this report, the volumetric ratio of FRP confinement was defined as the ratio of GFRP volume to the volume of confined concrete. For the GFRP tube with volumetric ratio of 1%, it can be found that the thickness of GFRP is approximately 0.45 mm.

The details of test specimens and the mechanical properties of longitudinal steel and transverse FRP reinforcement are listed in Table 2.12 to Table 2.14. Some information, such as the shear span of each column and the properties of GFRP, was not provided by Gould and Harmon (2002) but can be found in the related research report by Gould (1999). Only the range of concrete strength was mentioned in both reports and the average value of concrete strength is presented in Table 2.12 for columns in the two series.

### Table 2.12 Specimen details (Gould and Harmon, 2002)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Concrete strength, $f_c'$ (MPa)</th>
<th>Shear span, $L$ (mm)</th>
<th>Shear span-depth ratio</th>
<th>FRP tube volume ratio (%)</th>
<th>FRP confinement</th>
<th>Axial load level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$P$ (kN)</td>
<td>$P/P_c$</td>
</tr>
<tr>
<td>H2P30G0</td>
<td>44.8</td>
<td>521</td>
<td>2.9</td>
<td>0</td>
<td>-</td>
<td>133</td>
</tr>
<tr>
<td>H2P30G1</td>
<td>44.8</td>
<td>533</td>
<td>3.0</td>
<td>1</td>
<td>0.45 mm thick GFRP</td>
<td>133</td>
</tr>
<tr>
<td>H2P30G5</td>
<td>44.8</td>
<td>521</td>
<td>2.9</td>
<td>5</td>
<td>2.25 mm thick GFRP</td>
<td>133</td>
</tr>
<tr>
<td>H2P130G1</td>
<td>44.8</td>
<td>533</td>
<td>3.0</td>
<td>1</td>
<td>0.45 mm thick GFRP</td>
<td>578</td>
</tr>
<tr>
<td>H2P130G3</td>
<td>44.8</td>
<td>533</td>
<td>3.0</td>
<td>3</td>
<td>1.35 mm thick GFRP</td>
<td>578</td>
</tr>
<tr>
<td>H2P130G5</td>
<td>44.8</td>
<td>521</td>
<td>2.9</td>
<td>5</td>
<td>2.25 mm thick GFRP</td>
<td>578</td>
</tr>
<tr>
<td>H4P30G0</td>
<td>51.7</td>
<td>1146</td>
<td>6.4</td>
<td>0</td>
<td>-</td>
<td>133</td>
</tr>
<tr>
<td>H4P30G1</td>
<td>51.7</td>
<td>1156</td>
<td>6.4</td>
<td>1</td>
<td>0.45 mm thick GFRP</td>
<td>133</td>
</tr>
<tr>
<td>H4P30G5</td>
<td>51.7</td>
<td>1146</td>
<td>6.4</td>
<td>5</td>
<td>2.25 mm thick GFRP</td>
<td>133</td>
</tr>
<tr>
<td>H4P130G1</td>
<td>51.7</td>
<td>1156</td>
<td>6.4</td>
<td>1</td>
<td>0.45 mm thick GFRP</td>
<td>578</td>
</tr>
<tr>
<td>H4P130G3</td>
<td>51.7</td>
<td>1153</td>
<td>6.4</td>
<td>3</td>
<td>1.35 mm thick GFRP</td>
<td>578</td>
</tr>
<tr>
<td>H4P130G5</td>
<td>51.7</td>
<td>1146</td>
<td>6.4</td>
<td>5</td>
<td>2.25 mm thick GFRP</td>
<td>578</td>
</tr>
</tbody>
</table>
Table 2.13 Mechanical properties of steel reinforcement (Gould and Harmon, 2002)

<table>
<thead>
<tr>
<th>Reinforcement Bar Size</th>
<th>Diameter, $d_b$ (mm)</th>
<th>Area, $A_s$ ($\text{mm}^2$)</th>
<th>Yield strength, $f_y$ (MPa)</th>
<th>Elastic modulus, $E_s$ (MPa)</th>
<th>Yield strain, $\varepsilon_y$</th>
<th>Begin of strain-hardening, $\varepsilon_{sh}$</th>
<th>Ultimate strain, $\varepsilon_u$</th>
<th>Ultimate strength, $f_u$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal US#4</td>
<td>12.7</td>
<td>127</td>
<td>604</td>
<td>201000</td>
<td>0.0030</td>
<td>0.0126</td>
<td>0.162</td>
<td>749</td>
</tr>
</tbody>
</table>

Table 2.14 Mechanical properties of FRP composites (Gould and Harmon, 2002)

<table>
<thead>
<tr>
<th>FRP type</th>
<th>Tensile strength (MPa)</th>
<th>Rupture strain (mm/mm)</th>
<th>Elastic modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E-Glass FRP Tube</td>
<td>1379</td>
<td>0.020</td>
<td>68948</td>
</tr>
</tbody>
</table>

The hysteretic loops of moment vs. curvature and lateral shear vs. tip deflection curves were reported by Gould (1999) and Gould and Harmon (2002) for each column. The following points were made based on the test results:

1) In all the current design procedures, various uniaxial stress-strain models for confined concrete, generally developed from concentric compression tests, are applied directly to columns in bending. It implies that the strain gradient due to bending has no significant influence on the stress-strain relationship. This was referred to as the local strain model here. However, in this study it was found that the local strain model overestimated flexural stiffness by a small amount. Meanwhile, it underestimated the average wrap strain by up to a factor of two, even for monotonic eccentric loading without shear. More significantly, it greatly overestimated the peak wrap stress when columns were subjected to flexural and axial load, which led to large errors in predicting the ultimate failure point and ductility level of the confined columns.

2) In the existing design procedures, it is assumed that the maximum wrap strain is either due to shear or the combination of flexural and axial load. Interaction between shear and the combination of flexural and axial load is not considered (Seible et al. 1997). Both the analytical models and experiments indicate that the combination of shear, moment, and axial load can cause much higher wrap stress than any one of these forces acting alone. The current design codes which treat shear separately from confinement do not reflect the realistic behaviour of the FRP wrapping.
3) Axial compression restrains flexural and diagonal shear cracking and therefore increases the shear strength of columns in the case of monotonic loading. Under cyclic loading and high ductility demand, however, axial compression will likely increase the strains in FRP and then decrease shear strength of columns if shear failure is due to the failure of FRP wrap.

4) Most design codes do not consider the fracture of longitudinal steel as a possible failure mode. However most specimens in this program failed due to the low cycle fatigue failure of the longitudinal reinforcement under seismic loading and typically limited the ductility. It should be noted that the steel used in this study had higher strength and perhaps lower rupture strain than the steel commonly used. Therefore, this failure mode should be taken into account in further development of seismic design provisions.

5) In most design codes and guidelines such as the then FHWA design procedure and ACI 440.2R-08, the effective strain in all kinds of FRP jackets is limited to no more than 0.004 to ensure the shear integrity of the confined concrete. However, in this experiment, it was observed that the average strain in the GFRP tubes could be as high as 0.008 and the peak FRP strain as 0.02. Therefore, the more realistic limitation of effective FRP strain can be proposed to be a fraction of the fiber rupture strain.

6) Several models were proposed which predicted the average FRP strain, curvature, and displacements of the tested circular columns reasonably well when shear was not present. However, the predictions may have obvious errors in the columns with significant shear.

2.3.2.4 Iacobucci, Sheikh, and Bayrak (2003) and Memon and Sheikh (2005)

Iacobucci et al. (2003) and Memon and Sheikh (2005) conducted experimental research to investigate the effectiveness of external FRP reinforcement to strengthen substandard and repair damaged square concrete columns. These two investigations belonged to the same comprehensive research program, which were similar in all respects except that CFRP jackets were used in the tests by Iacobucci et al. (2003), whereas GFRP jackets were used by Memon and Sheikh (2005). The geometric properties of specimens, test setup and loading sequence were similar to those in the work of Sheikh and Khoury (1993).
Sixteen normal strength concrete square columns were tested under lateral cyclic loads, while subjected to constant axial load simultaneously. Among all the columns, two were control specimens which had only widely spaced transverse steel ties and were not retrofitted by any FRP wrapping. Ten columns had initially only substandard transverse steel ties and were then strengthened by external CFRP or GFRP wrapping. The remaining four substandard columns were first loaded to a certain level of damage and then repaired by external FRP wrapping while still subjected to certain level of axial load. Each of the sixteen specimens consisted of a square column with a 305×305 mm cross section and a 1473 mm height, attached to a 508×763×813 mm heavily reinforce stub. Each column contained eight 20M longitudinal steel bars ($\rho_g = 2.58\%$) and #3@300 mm transverse ties of Type A configuration ($\rho_s = 0.61\%$), as shown in Figure 2.4. The concrete core was 267×267 mm, measured to the centerline of the peripheral ties. Unidirectional carbon or glass fibers in the FRP jackets were aligned in the circumferential direction of the columns to provide only external transverse reinforcement. The corners of the columns were rounded to a 16 mm radius before wrapping the FRP to minimize the premature failure of FRP jackets due to stress concentration.

The shear span of each column was 1841 mm and thus the shear span-depth ratio was 6.04. The details of the test specimens and mechanical properties of steel and FRP reinforcement are introduced in Table 2.15 to Table 2.17.

The experimental results were reported in the forms of hysteresis loops of moment vs. curvature and lateral shear vs. tip deflection curves. The following conclusions were drawn:

1) Ductility and energy dissipation capacity of the deficient square columns can be improved significantly by transverse CFRP and GFRP jackets. In these tests, the seismic behaviour of properly FRP-retrofitted square specimens was superior to that of similar columns in which steel-confine ment was provided in accordance with the seismic provisions of the then current codes (ACI 318-02 and CAN/CSA-A23.3-94).

2) The required amount of FRP confinement had to be increased significantly with the increase of axial load level for columns to obtain a certain level of curvature ductility. Hence, it was stressed that the design of FRP confinement of the columns should take into account the effect of axial load level.
### Table 2.15 Specimen details (Iacobucci et al., 2003, and Memon and Sheikh, 2005)

<table>
<thead>
<tr>
<th>Researcher</th>
<th>Specimen</th>
<th>Concrete strength, $f_c$ (MPa)</th>
<th>Bar size</th>
<th>$s$ (mm)</th>
<th>$\rho_s$ (%)</th>
<th>$A_{sh}/A_{sh,ACI}$</th>
<th>FRP confinement</th>
<th>Axial load level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Iacobucci et al. (2003)</td>
<td>AS-1NS</td>
<td>31.4</td>
<td>US #3</td>
<td>300</td>
<td>0.61</td>
<td>0.49</td>
<td>-</td>
<td>1166</td>
</tr>
<tr>
<td> </td>
<td>ASC-2NS</td>
<td>36.5</td>
<td>US #3</td>
<td>300</td>
<td>0.61</td>
<td>0.42</td>
<td>1-layer 1.00 mm CFRP</td>
<td>1296</td>
</tr>
<tr>
<td> </td>
<td>ASC-3NS</td>
<td>36.9</td>
<td>US #3</td>
<td>300</td>
<td>0.61</td>
<td>0.42</td>
<td>2-layer 1.00 mm CFRP</td>
<td>2217</td>
</tr>
<tr>
<td> </td>
<td>ASC-4NS</td>
<td>36.9</td>
<td>US #3</td>
<td>300</td>
<td>0.61</td>
<td>0.42</td>
<td>1-layer 1.00 mm CFRP</td>
<td>2217</td>
</tr>
<tr>
<td> </td>
<td>ASC-5NS</td>
<td>37.0</td>
<td>US #3</td>
<td>300</td>
<td>0.61</td>
<td>0.42</td>
<td>3-layer 1.00 mm CFRP</td>
<td>2221</td>
</tr>
<tr>
<td> </td>
<td>ASC-6NS</td>
<td>37.0</td>
<td>US #3</td>
<td>300</td>
<td>0.61</td>
<td>0.42</td>
<td>2-layer 1.00 mm CFRP</td>
<td>1309</td>
</tr>
<tr>
<td> </td>
<td>ASC-7NS</td>
<td>37.0</td>
<td>US #3</td>
<td>300</td>
<td>0.61</td>
<td>0.42</td>
<td>-</td>
<td>1309</td>
</tr>
<tr>
<td> </td>
<td>ASC-8NS</td>
<td>42.3</td>
<td>US #3</td>
<td>300</td>
<td>0.61</td>
<td>0.42</td>
<td>-</td>
<td>2467</td>
</tr>
<tr>
<td>Memon and Sheikh (2005)</td>
<td>AS-1NSS</td>
<td>42.4</td>
<td>US #3</td>
<td>300</td>
<td>0.61</td>
<td>0.36</td>
<td>-</td>
<td>2500</td>
</tr>
<tr>
<td> </td>
<td>ASG-2NSS</td>
<td>42.4</td>
<td>US #3</td>
<td>300</td>
<td>0.61</td>
<td>0.36</td>
<td>2-layer 1.25 mm GFRP</td>
<td>1490</td>
</tr>
<tr>
<td> </td>
<td>ASG-3NSS</td>
<td>42.7</td>
<td>US #3</td>
<td>300</td>
<td>0.61</td>
<td>0.36</td>
<td>4-layer 1.25 mm GFRP</td>
<td>2500</td>
</tr>
<tr>
<td> </td>
<td>ASG-4NSS</td>
<td>43.3</td>
<td>US #3</td>
<td>300</td>
<td>0.61</td>
<td>0.35</td>
<td>2-layer 1.25 mm GFRP</td>
<td>2500</td>
</tr>
<tr>
<td> </td>
<td>ASG-5NSS</td>
<td>43.7</td>
<td>US #3</td>
<td>300</td>
<td>0.61</td>
<td>0.35</td>
<td>1-layer 1.25 mm GFRP</td>
<td>1490</td>
</tr>
<tr>
<td> </td>
<td>ASG-6NSS</td>
<td>44.2</td>
<td>US #3</td>
<td>300</td>
<td>0.61</td>
<td>0.34</td>
<td>6-layer 1.25 mm GFRP</td>
<td>2500</td>
</tr>
<tr>
<td> </td>
<td>AS-7NSS</td>
<td>44.2</td>
<td>US #3</td>
<td>300</td>
<td>0.61</td>
<td>0.34</td>
<td>-</td>
<td>1490</td>
</tr>
<tr>
<td> </td>
<td>AS-8NSS</td>
<td>44.2</td>
<td>US #3</td>
<td>300</td>
<td>0.61</td>
<td>0.34</td>
<td>-</td>
<td>2500</td>
</tr>
</tbody>
</table>

### Table 2.16 Mechanical properties of steel reinforcement (Iacobucci et al., 2003, and Memon and Sheikh, 2005)

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Bar size</th>
<th>Diameter, $d_b$ (mm)</th>
<th>Area, $A_s$ (mm$^2$)</th>
<th>Yield strength, $f_y$ (MPa)</th>
<th>Elastic modulus, $E_s$ (MPa)</th>
<th>Yield strain, $\varepsilon_y$</th>
<th>Begin of strain-hardening, $\varepsilon_{sh}$</th>
<th>Ultimate strain, $\varepsilon_u$</th>
<th>Ultimate strength, $f_u$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse US#3</td>
<td>9.53</td>
<td>71.3</td>
<td>457</td>
<td>207730</td>
<td>0.0022</td>
<td>0.0070</td>
<td>0.1050</td>
<td>739</td>
<td></td>
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<tr>
<td>Longitudinal 20M</td>
<td>19.54</td>
<td>300</td>
<td>465</td>
<td>202170</td>
<td>0.0023</td>
<td>0.0113</td>
<td>0.1288</td>
<td>640</td>
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</tr>
</tbody>
</table>

### Table 2.17 Mechanical properties of FRP composites (Iacobucci et al., 2003, and Memon and Sheikh, 2005)

<table>
<thead>
<tr>
<th>FRP type</th>
<th>Nominal thickness (mm/ply)</th>
<th>Tensile strength (MPa)</th>
<th>Rupture strain (mm/mm)</th>
<th>Elastic modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP</td>
<td>1.00</td>
<td>962</td>
<td>0.0126</td>
<td>76349</td>
</tr>
<tr>
<td>GFRP</td>
<td>1.25</td>
<td>450</td>
<td>0.0228</td>
<td>19737</td>
</tr>
</tbody>
</table>
3) It was found that shear and moment capacities of columns were also enhanced by the FRP confinement. The moment vs. curvature envelopes of the FRP-confined columns showed stable post-elastic branches without obvious strength degradation. This observation is very different from the comparable columns solely confined by transverse steel reinforcement.

4) Pre-damaged specimens were rehabilitated effectively by external FRP wrapping. The required amount of the FRP also depends on the extent of damage severity.

5) The confinement mechanism in square columns, because of its shape, is less efficient than in circular columns. Thus, FRP-retrofitted square columns experienced less seismic improvement in terms of ductility and energy dissipation capacity than comparable FRP-retrofitted circular columns.

2.3.2.5 Hosseini, Khaloo, and Fadaee (2005)

This paper presents the experimental results of six high-strength concrete square columns with different levels of initial ductility, in which two specimens were strengthened by CFRP wraps. Each column had a 260×260 mm cross section and a length of 1650 mm, cast integrally with a 400×500×900 mm stub. The columns were tested under lateral cyclic loading, while simultaneously subjected to a constant axial load throughout the test. The shear span was 1500 mm measured from the column base to the point of application of lateral load.

The six specimens were divided into three groups of two columns each according to the different confinement mechanism and level. The specimens in Groups I and II were initially designed and detailed according to the requirements of ACI318-02 for intermediate-moment frames. The two columns in Group I were then wrapped with three layers of CFRP jackets before testing, while group II were control specimens and tested without any FRP retrofit. Specimens in group III were designed and detailed according to the requirements of ACI318-02 for special-moment frames. The two columns in each group had four and eight 18 mm diameter G400 deformed longitudinal reinforcement (\(\rho_{sl} = 1.5\% \text{ and } 3.0\%)\), respectively. Clear concrete cover was 25 mm in all columns, measured to the outer surface of the steel ties.

The corners of two square columns in Group I were rounded to a radius of 15 mm before CFRP wrapping. The unidirectional fibers in the continuous CFRP sheets were aligned along the
circumferential direction of the columns to provide only external transverse reinforcement. The details of the test specimens and mechanical properties of steel and FRP reinforcement are introduced in Table 2.18 and Table 2.19. Detailed properties of Grade 400 steel bars used for the longitudinal and transverse reinforcement were not provided in the paper.

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen</th>
<th>$f_{c'}$ (MPa)</th>
<th>$d_b$ (mm)</th>
<th>$\rho_d$ (%)</th>
<th>$d_b$ (mm)</th>
<th>$s$ (mm)</th>
<th>$\rho_t$ (%)</th>
<th>FRP confinement</th>
<th>$P$ (kN)</th>
<th>$P_o / P$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>WI4</td>
<td>53</td>
<td>4</td>
<td>18</td>
<td>1.5</td>
<td>10</td>
<td>120</td>
<td>1.24</td>
<td>3-layer CFRP</td>
<td>521</td>
</tr>
<tr>
<td></td>
<td>WI8</td>
<td>52</td>
<td>8</td>
<td>18</td>
<td>3.0</td>
<td>10</td>
<td>120</td>
<td>1.80</td>
<td>3-layer CFRP</td>
<td>577</td>
</tr>
<tr>
<td>II</td>
<td>CI4</td>
<td>56</td>
<td>4</td>
<td>18</td>
<td>1.5</td>
<td>10</td>
<td>120</td>
<td>1.24</td>
<td>-</td>
<td>546</td>
</tr>
<tr>
<td></td>
<td>CI8</td>
<td>54</td>
<td>8</td>
<td>18</td>
<td>3.0</td>
<td>10</td>
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<td>-</td>
<td>593</td>
</tr>
<tr>
<td>III</td>
<td>CS4</td>
<td>54</td>
<td>4</td>
<td>18</td>
<td>1.5</td>
<td>12</td>
<td>70</td>
<td>3.10</td>
<td>-</td>
<td>529</td>
</tr>
<tr>
<td></td>
<td>CS8</td>
<td>53</td>
<td>8</td>
<td>18</td>
<td>3.0</td>
<td>10</td>
<td>70</td>
<td>3.20</td>
<td>-</td>
<td>585</td>
</tr>
</tbody>
</table>

Table 2.19 Mechanical properties of FRP composites (Hosseini et al., 2005)

<table>
<thead>
<tr>
<th>Material</th>
<th>Nominal thickness (mm/ply)</th>
<th>Tensile strength (MPa)</th>
<th>Rupture strain (mm/mm)</th>
<th>Elastic modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP</td>
<td>0.16</td>
<td>3500</td>
<td>0.015</td>
<td>230 000</td>
</tr>
<tr>
<td>Epoxy</td>
<td>-</td>
<td>70</td>
<td>-</td>
<td>10 000</td>
</tr>
</tbody>
</table>

The hysteresis loops of moment vs. curvature and lateral shear vs. tip deflection curves have been reported by Hosseini et al. (2005) for each column. The following conclusions were drawn based on the results:

1) Compared to the relevant control specimens, the displacement and curvature ductility factors in column WI4 were improved for 53% and 79%, respectively, by the CFRP-confinement. In column WI8, the improvements in the displacement and curvature ductility factors were 27% and 28%, respectively. Moreover, seismic performance of the two CFRP-retrofitted columns was enhanced to a level higher than that of the unwrapped columns designed according to seismic provisions for special-moment frames.
2) The unjacketed columns failed by the crushing of core concrete in the plastic hinge regions, while the FRP-retrofitted columns failed due to rupture of the CFRP jackets with significant enhanced strength and ductility.

3) The strain in the CFRP jackets began to increase rapidly when the column lateral displacement was beyond $\mu_\Delta = 3$. At close to failure the measured jacket strain was less than one half of the ultimate strain measured in the FRP coupon tension tests.

2.3.2.6 Ozbakkaloglu and Saatcioglu (2006, 2007)

Seismic tests were reported by Ozbakkaloglu and Saatcioglu (2006, 2007) for ten high-strength concrete columns transversely confined by prefabricated FRP tubes. The main variables were concrete strength, axial load, shear spans, shapes of cross section and amount of CFRP-confinement. Each of the four 270 mm diameter circular columns and the six 270×270 mm square columns was cast integrally with a 500×500×1100 mm stub and had a shear span of 2000 mm measured from the column base to the point of application of lateral load, except that the shear span of specimen RC-4 was 1200 mm. All columns were tested under lateral cyclic loading, while simultaneously subjected to a constant axial load throughout testing.

None of the columns had transverse steel reinforcement. Prefabricated CFRP circular or square tubes were used as stay-in-place formwork which also provided transverse confining reinforcement. The corners of the formworks were rounded to a radius of 45 mm for all square columns, except for a radius of 8 mm in column RS-6. The unidirectional fibers in the CFRP tubes were aligned along the circumferential direction of columns to provide only external transverse reinforcement. FRP crossties were used in square columns RS-2, RS-3 and RS-5 to improve the confinement efficiency of square FRP-tubes. Clear concrete cover was 25 mm in all columns, measured to the outer surface of the longitudinal steel reinforcement. The details of the test specimens and mechanical properties of steel and FRP-reinforcement are provided in Table 2.20 to Table 2.22.

The hysteresis loops of moment vs. lateral drift/displacement ductility relationship were reported in these two papers for each column, rather than the moment vs. curvature or shear-
displacement relationships which are commonly presented by other researchers. The reported experimental results of displacement ductility and lateral drift ratios were determined based on the moment vs. lateral displacement curves, while no curvature ductility factor was reported for any specimen. Thus, the detrimental $P$-$\Delta$ effect to the displacement ductility and lateral drift capacity of columns were not included in the reported experimental results.

Table 2.20 Specimen details (Ozbakkaloglu1 and Saatcioglu, 2006, 2007)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$f'_{c}$ (MPa)</th>
<th>Shear span (mm)</th>
<th>Longitudinal steel reinforcement</th>
<th>Number of FRP layers in tubes</th>
<th>Axial load level $P$ (kN)</th>
<th>$P/oP$</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC-1</td>
<td>90.1</td>
<td>2000</td>
<td>8</td>
<td>2.79</td>
<td>4</td>
<td>1580</td>
</tr>
<tr>
<td>RC-2</td>
<td>75.2</td>
<td>2000</td>
<td>8</td>
<td>2.79</td>
<td>2</td>
<td>1480</td>
</tr>
<tr>
<td>RC-3</td>
<td>49.7</td>
<td>2000</td>
<td>8</td>
<td>2.79</td>
<td>2</td>
<td>1480</td>
</tr>
<tr>
<td>RC-4</td>
<td>75.3</td>
<td>1200</td>
<td>8</td>
<td>2.79</td>
<td>2</td>
<td>1480</td>
</tr>
<tr>
<td>RS-1</td>
<td>90.1</td>
<td>2000</td>
<td>4</td>
<td>19.5</td>
<td>1.68</td>
<td>5</td>
</tr>
<tr>
<td>RS-2</td>
<td>90.1</td>
<td>2000</td>
<td>8</td>
<td>16.0</td>
<td>2.24</td>
<td>5</td>
</tr>
<tr>
<td>RS-3</td>
<td>90.1</td>
<td>2000</td>
<td>12</td>
<td>2.79</td>
<td>3.36</td>
<td>5</td>
</tr>
<tr>
<td>RS-4</td>
<td>75.2</td>
<td>2000</td>
<td>8</td>
<td>16.0</td>
<td>2.24</td>
<td>3</td>
</tr>
<tr>
<td>RS-5</td>
<td>75.2</td>
<td>2000</td>
<td>8</td>
<td>2.24</td>
<td>2</td>
<td>1760</td>
</tr>
<tr>
<td>RS-6</td>
<td>75.2</td>
<td>2000</td>
<td>8</td>
<td>2.20</td>
<td>3</td>
<td>1800</td>
</tr>
</tbody>
</table>

Table 2.21 Mechanical properties of reinforcing steels (Ozbakkaloglu1 and Saatcioglu, 2006, 2007)

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Diameter, $d_b$ (mm)</th>
<th>Area, $A_s$ (mm$^2$)</th>
<th>Yield strength, $f_y$ (MPa)</th>
<th>Yield strain, $\varepsilon_y$</th>
<th>Elastic modulus, $E_s$ (MPa)</th>
<th>Ultimate strength, $f_u$ (MPa)</th>
<th>Ultimate strain, $\varepsilon_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal</td>
<td>16.0</td>
<td>200</td>
<td>500</td>
<td>0.0024</td>
<td>208750</td>
<td>620</td>
<td>0.120</td>
</tr>
<tr>
<td></td>
<td>19.5</td>
<td>300</td>
<td>476</td>
<td>0.0026</td>
<td>182840</td>
<td>570</td>
<td>0.125</td>
</tr>
</tbody>
</table>

Table 2.22 Mechanical properties of FRP composites (Ozbakkaloglu1 and Saatcioglu, 2006, 2007)

<table>
<thead>
<tr>
<th>Material</th>
<th>Nominal thickness (mm/ply)</th>
<th>Tensile strength (MPa)</th>
<th>Rupture strain</th>
<th>Elastic modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon FRP</td>
<td>0.165</td>
<td>3800</td>
<td>0.0167</td>
<td>227 000</td>
</tr>
</tbody>
</table>
Following conclusions were drawn by the authors based on test results in the papers:

1) High-strength concrete circular and square column specimens confined by CFRP stay-in-place formwork can develop extremely ductile behaviour under simulated seismic loading. Reported in these experimental programs, the lateral drift ratio of both circular and square columns with concrete of $f_c' = 90$ MPa can be improved up to higher than 11% by FRP tubes.

2) FRP tubes provide continuous confinement to the entire column area, thus resulting in higher confinement efficiency comparing to conventional steel reinforcement cages which only confine the core concrete in a discrete pattern.

3) During seismic tests, the measured strain of CFRP tubes in hoop direction could be as high as 1.5%, which approaches the tensile capacity of the FRP material. It indicated that limiting the effective FRP strain to be no more than 0.4% in some FRP design approaches is over-conservative.

4) The confinement efficiency of square FRP-tubes can be significantly improved by the increase of the ratio of corner radius to column dimension and the use of FRP crossties.

2.3.2.7 Wang, Lu, Li, and Wang (2009)

Eight circular columns were tested under constant axial load and cyclic lateral force to investigate their seismic performance and the effectiveness of confinement by the CFRP jackets. A distinct characteristic of this experimental research is that all the columns were constructed of high strength concrete and subjected to high axial load ratios.

All the columns had the same geometric properties and steel reinforcement. Each column had a diameter of 180 mm and a height of 1260 mm. During testing, the specimens were subjected to double bending and the shear span-depth ratio was 3.5. Six 12 mm diameter deformed steel bars were used as longitudinal reinforcement, while circular hoops of 4 mm @60 mm were used as transverse reinforcement. The clear concrete cover was reported as 8 mm, which was deemed to be measured to the outer surface of longitudinal steel according to the conventional definition of concrete cover in the country where the tests were conducted.
Among all the specimens, six columns were confined with four layers of CFRP jackets, while the other two control specimens had no FRP wrapping. The fibers of CFRP were aligned along the circumferential direction of columns and no strength contribution from FRP was expected in the longitudinal direction of columns.

The details of specimens and the material properties are shown in Table 2.23 to Table 2.25. The originally reported concrete cube strength $f_{cu}$ of each column was converted into the cylinder strength $f'_c$ of concrete according to the recommendation by fib Model Code (2010).

### Table 2.23 Tests information (Wang et al., 2009)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Concrete cube strength, $f_{cu}$ (MPa)</th>
<th>Concrete cylinder strength, $f'_c$ (MPa)</th>
<th>FRP confinement</th>
<th>Axial load</th>
<th>$P/(Afg_{cu})$</th>
<th>$P/P_o$</th>
<th>$P$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C60N1_U</td>
<td>56.2</td>
<td>47.7</td>
<td></td>
<td></td>
<td>0.45</td>
<td>0.52</td>
<td>644</td>
</tr>
<tr>
<td>C60N1_F</td>
<td>56.2</td>
<td>47.7</td>
<td>4-layer CFRP</td>
<td></td>
<td>0.45</td>
<td>0.52</td>
<td>644</td>
</tr>
<tr>
<td>C60N2_F</td>
<td>56.2</td>
<td>47.7</td>
<td>4-layer CFRP</td>
<td></td>
<td>0.55</td>
<td>0.63</td>
<td>787</td>
</tr>
<tr>
<td>C60N3_F</td>
<td>56.2</td>
<td>47.7</td>
<td>4-layer CFRP</td>
<td></td>
<td>0.65</td>
<td>0.75</td>
<td>930</td>
</tr>
<tr>
<td>C80N1_U</td>
<td>72.9</td>
<td>64.1</td>
<td></td>
<td></td>
<td>0.45</td>
<td>0.53</td>
<td>835</td>
</tr>
<tr>
<td>C80N1_F</td>
<td>72.9</td>
<td>64.1</td>
<td>4-layer CFRP</td>
<td></td>
<td>0.45</td>
<td>0.53</td>
<td>835</td>
</tr>
<tr>
<td>C80N2_F</td>
<td>72.9</td>
<td>64.1</td>
<td>4-layer CFRP</td>
<td></td>
<td>0.55</td>
<td>0.64</td>
<td>1020</td>
</tr>
<tr>
<td>C80N3_F</td>
<td>72.9</td>
<td>64.1</td>
<td>4-layer CFRP</td>
<td></td>
<td>0.65</td>
<td>0.76</td>
<td>1206</td>
</tr>
</tbody>
</table>

### Table 2.24 Mechanical properties of reinforcing steels (Wang et al., 2009)

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Diameter, $d_b$ (mm)</th>
<th>Area, $A_s$ (mm²)</th>
<th>Yield strength, $f_y$ (MPa)</th>
<th>Yield strain, $\varepsilon_y$</th>
<th>Elastic modulus, $E_s$ (MPa)</th>
<th>Ultimate strength, $f_u$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse</td>
<td>4</td>
<td>12.57</td>
<td>402</td>
<td>0.00201</td>
<td>200000</td>
<td>808</td>
</tr>
<tr>
<td>Longitudinal</td>
<td>10</td>
<td>113.1</td>
<td>353</td>
<td>0.00177</td>
<td>200000</td>
<td>516</td>
</tr>
</tbody>
</table>

### Table 2.25 Mechanical properties of FRP composites (Wang et al., 2009)

<table>
<thead>
<tr>
<th>Material</th>
<th>Nominal thickness (mm/ply)</th>
<th>Tensile strength (MPa)</th>
<th>Rupture strain</th>
<th>Elastic modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon FRP</td>
<td>0.167</td>
<td>3430</td>
<td>0.0149</td>
<td>230 000</td>
</tr>
</tbody>
</table>
The test results were presented as the hysteresis loops of lateral shear vs. tip deflection of each column by Wang et al. (2009). The effects of axial load ratio, FRP confinement and concrete strength on the behaviour and energy dissipation capacity of columns were investigated. The following conclusions were drawn based on the test results:

1) FRP wrapping at the plastic hinge zone of the column could significantly improve its ductility, energy dissipation capacity and seismic resistance. The six FRP-retrofitted columns displayed ductile behaviour with lateral drift capacity of about 6%, while the two control specimens failed in a very brittle manner with drift ratio of 1.5% - 2.0%.

2) It was proposed that the FRP jackets need to be used only within the plastic hinge zone of 1.8D length \( (D = \text{diameter of circular column}) \) for columns with shear span ratio greater than 3.0.

3) It was also found that the strain development in the internal transverse steel lagged behind the strain in the FRP wrapping, so that the confinement was mostly provided by the FRP wrapping.

### 2.3.3 Summary of Experimental Work

Selected experimental studies have been presented here in which all specimens were transversely confined by steel reinforcement or FRP wrapping and tested under simulated earthquake loading i.e. under laterally reversed cyclic load while simultaneously subjected to constant axial load. The summary of these experiments are presented in Table 2.26 and Table 2.27. The experimental results were reported in the form of hysteresis loops of moment vs. curvature and lateral shear vs. tip deflection curves in the studies, except the work reported by Ozbakkaloglu and Saatcioglu (2006, 2007). The published work will be used to verify the analytical program developed in Chapter 5.

Based on the reported experimental results, several common conclusions are drawn although the details of tests were different from one program to another.

1) The seismic resistance of the concrete columns can be dramatically improved by providing transverse steel or FRP-confinement. Ductility, deformability, energy dissipation capacity
and flexural strength of the concrete columns are enhanced with the increase of transverse confinement level.

2) The required amount of transverse confinement should be increased with the increase of axial load level to achieve similar curvature ductility level.

3) Seismic response behaviour of steel-confined columns differs distinctly from that of FRP-confined columns. FRP-confined columns have almost no degradation of flexural strength in the plastic deformation stage, while steel-confined columns may perform a post-peak branch of moment vs. curvature relationship. This diversity is even clearer for columns under higher level of axial load.

4) The efficiency of transverse confinement is much higher in the columns with circular cross section than in the columns with square cross section.

5) Transverse confinement can lead to significant enhancement of flexural strength in columns under high level of axial loading.

6) The required amount of transverse confinement may be very different if different ductility parameters are used as design criteria. This is because different relationships were observed between the transverse confinement and various ductility parameters, such as curvature ductility factor, displacement ductility factor and lateral drift ratio in the seismic tests on concrete columns.
<table>
<thead>
<tr>
<th>Researcher</th>
<th>Numbers of Specimens</th>
<th>Column length (mm)</th>
<th>Section type</th>
<th>Cross section size, D (mm)</th>
<th>Axial load level, P/Po</th>
<th>Concrete strength, fc' (MPa)</th>
<th>Transverse confinement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheikh and Khoury (1993)</td>
<td>6</td>
<td>1473</td>
<td>square</td>
<td>305</td>
<td>0.39-0.63</td>
<td>31.3-33.2</td>
<td>ties</td>
</tr>
<tr>
<td>Sheikh et al. (1994)</td>
<td>3</td>
<td>1473</td>
<td>square</td>
<td>305</td>
<td>0.59-0.62</td>
<td>53.6-59.1</td>
<td>ties</td>
</tr>
<tr>
<td>Bayrak and Sheikh (1998)</td>
<td>8</td>
<td>1473</td>
<td>square</td>
<td>305</td>
<td>0.36-0.50</td>
<td>71.7-102.2</td>
<td>ties</td>
</tr>
<tr>
<td>Yau and Sheikh (1998)</td>
<td>4</td>
<td>1473</td>
<td>circular</td>
<td>356</td>
<td>0.27-0.54</td>
<td>39.2-40.1</td>
<td>spirals</td>
</tr>
<tr>
<td>Iacobucci et al. (2003)</td>
<td>3</td>
<td>1473</td>
<td>square</td>
<td>305</td>
<td>0.33-0.56</td>
<td>31.4-37.0</td>
<td>ties</td>
</tr>
<tr>
<td>Memon and Sheikh (2005)</td>
<td>3</td>
<td>1473</td>
<td>square</td>
<td>305</td>
<td>0.33-0.56</td>
<td>42.4</td>
<td>ties</td>
</tr>
<tr>
<td>Bae (2005)</td>
<td>4</td>
<td>3050</td>
<td>square</td>
<td>440-610</td>
<td>0.20-0.50</td>
<td>36.5-43.4</td>
<td>ties</td>
</tr>
<tr>
<td>Hosseini et al. (2005)</td>
<td>4</td>
<td>1650</td>
<td>square</td>
<td>260</td>
<td>0.15</td>
<td>52.5</td>
<td>ties</td>
</tr>
<tr>
<td>Paultre et al. (2009)</td>
<td>6</td>
<td>2000</td>
<td>circular</td>
<td>300</td>
<td>0.16-0.43</td>
<td>97-109</td>
<td>spirals</td>
</tr>
<tr>
<td>Liu and Sheikh (2009)</td>
<td>8</td>
<td>1473</td>
<td>circular</td>
<td>356</td>
<td>0.27-0.56</td>
<td>40.0</td>
<td>spirals</td>
</tr>
<tr>
<td>Wang et al. (2009)</td>
<td>2</td>
<td>1260</td>
<td>circular</td>
<td>180</td>
<td>0.43-0.47</td>
<td>56.2-72.9</td>
<td>spirals</td>
</tr>
</tbody>
</table>

Table 2.27 FRP-confined column tests

<table>
<thead>
<tr>
<th>Researcher</th>
<th>Numbers of Specimens</th>
<th>Column length (mm)</th>
<th>Section type</th>
<th>Cross section size, D (mm)</th>
<th>Axial load level, P/Po</th>
<th>Concrete strength, fc' (MPa)</th>
<th>Transverse confinement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheikh and Yau (2002)</td>
<td>5</td>
<td>1473</td>
<td>circular</td>
<td>356</td>
<td>0.27-0.54</td>
<td>40.4-44.8</td>
<td>CFRP- and GFRP-jackets</td>
</tr>
<tr>
<td>Iacobucci et al. (2003)</td>
<td>5</td>
<td>1473</td>
<td>square</td>
<td>305</td>
<td>0.33-0.54</td>
<td>31.4-37.0</td>
<td>CFRP-jackets</td>
</tr>
<tr>
<td>Gould and Harmon (2002)</td>
<td>10</td>
<td>600-1200</td>
<td>circular</td>
<td>180</td>
<td>0.08-0.36</td>
<td>44.1-55.2</td>
<td>GFRP-tube</td>
</tr>
<tr>
<td>Memon and Sheikh (2005)</td>
<td>5</td>
<td>1473</td>
<td>square</td>
<td>305</td>
<td>0.33-0.56</td>
<td>42.5-44.2</td>
<td>GFRP-jackets</td>
</tr>
<tr>
<td>Hosseini et al. (2005)</td>
<td>2</td>
<td>1650</td>
<td>square</td>
<td>260</td>
<td>0.15</td>
<td>52.5</td>
<td>CFRP-jackets</td>
</tr>
<tr>
<td>Ozbakkaloglu and Saatcioglu (2006, 2007)</td>
<td>10</td>
<td>1200-2000</td>
<td>Square/circular</td>
<td>270</td>
<td>0.30-0.47</td>
<td>49.7-90.1</td>
<td>CFRP-tubes</td>
</tr>
<tr>
<td>Wang et al. (2009)</td>
<td>6</td>
<td>1260</td>
<td>circular</td>
<td>180</td>
<td>0.43-0.68</td>
<td>56.2-72.9</td>
<td>CFRP-jackets</td>
</tr>
<tr>
<td>Liu and Sheikh (2009)</td>
<td>7</td>
<td>1473</td>
<td>circular</td>
<td>356</td>
<td>0.27-0.56</td>
<td>40.0</td>
<td>CFRP- and GFRP-jackets</td>
</tr>
</tbody>
</table>
2.4 Design of confining reinforcement in columns

2.4.1 Introduction

Different seismic design codes propose very different provisions for the design of transverse reinforcement owing to the lack of sufficient information and uniform design criteria and perhaps a lack of agreement among researchers. In this section, design provisions for confining steel reinforcement in various design codes (ACI 318-11, CAN/CSA-A23.3-04, EN 1998-1:2004 and NZS 3101:2006) are reviewed and discussed. To better understand these code provisions, the early background research work that led to the code development efforts is also briefly introduced prior to the review of each code provision.

The development of design codes for seismic retrofit of columns with external FRP jackets is still lagging behind the engineering practice mainly due to the limited research results. For instance, there is no design procedure in CAN/CSA-S6-06 to determine the amount of FRP confinement in columns for certain ductility target, while only a principle guideline was included in ACI 440.2R-08 for seismic retrofit of columns. Only in CAN/CSA-S806-12, design provisions for the seismic retrofit of concrete columns by FRP confinement are provided, which will be discussed later in this chapter.

A series of systematic design procedures have been developed by Sheikh and Khoury (1997), Bayrak and Sheikh (1998) and Sheikh and Li (2005) to determine the demand of transverse steel or FRP confinement for concrete columns to achieve certain levels of curvature ductility.

Besides providing confinement to the concrete core, transverse reinforcement in columns has two other important functions: preventing the premature buckling of longitudinal reinforcement and providing sufficient shear resistance. For instance, to prevent the buckling of longitudinal reinforcement, it is required in most seismic design codes that the spacing of spirals or ties shall not exceed the smaller of 1/4 of the least dimension of the cross section or 6 times the diameter of the longitudinal bar to be restrained in a ductile concrete column. An expression is further proposed in NZS 3101:2006 to determine the required minimum area of a leg of ties to prevent the buckling of longitudinal reinforcement. These two functions of the transverse reinforcement are beyond the scope of this research and only the issues about confinement are discussed in the following sections.
2.4.2 ACI 318-11

Design provisions in the Clauses 10.9 and 21.6.4 of ACI 318-11 to determine the required transverse reinforcement for concrete columns are based on the relationship between the enhancement of compressive strength of concrete and transverse confinement, in which the axial load capacity of columns is taken as the design criterion.

The relationship between the enhancement of compressive strength and the transverse confinement of concrete was proposed by Richart et al. (1928, 1929) based on the experimental results of normal-strength concrete transversely confined by spirals or hydraulic pressure. Although no expression was suggested in these two papers for the full stress-strain relationship of transversely confined concrete, the evaluation of compressive strength $f'_{cc}$ and the corresponding strain $\varepsilon_{cc}$ were recommended as follows:

$$
f'_{cc} = f'_c + 4.1 f_i \tag{2-41}
$$

$$
\varepsilon_{cc} = \varepsilon_c \left(1 + 20.5 \frac{f_i}{f'_c} \right) \tag{2-42}
$$

where:

- $f'_c$ = 28-day cylinder strength of unconfined concrete;
- $\varepsilon_c$ = compressive strain corresponding to $f'_c$;
- $f_i$ = transverse confining stress.

Based on expression (2-41), design provisions were developed for ACI 318 to determine the required transverse reinforcement in concrete columns. The goal was that the strength loss due to the spalling of concrete cover should be compensated for by the strength enhancement of the concrete core resulting from the transverse confinement, so that the original axial load carrying capacity of the column can be maintained. This rationale behind the code provisions has not changed since the 1930s. The volumetric ratio $\rho_s$ of spiral or circular hoop reinforcement in potential plastic hinges of concrete columns shall not be less than the values determined by the following two equations:

$$
\rho_s = 0.45 \left( \frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f'_{yh}} \tag{2-43}
$$
\[ \rho_s = 0.12 \frac{f_c'}{f_{yh}} \]  (2-44)

Recognizing the lower effectiveness of confinement by rectilinear ties, the above two equations were extended for square and rectangular columns, in which the total effective area \( A_{sh} \) of transverse reinforcement in the direction under consideration shall be no less than the requirements by the following two equations:

\[ A_{sh} = 0.3b_c \left( \frac{A_g}{A_{ch}} - 1 \right) \frac{f_c'}{f_{yh}} \]  (2-45)

\[ A_{sh} = 0.09b_c \frac{f_c'}{f_{yh}} \]  (2-46)

where:

- \( A_g \) = gross area of the cross section of column;
- \( A_{ch} \) = cross-sectional area of the concrete core measured to the outer surface of peripheral transverse reinforcement;
- \( f_c' \) = specified compressive strength of concrete;
- \( f_{yh} \) = yield strength of transverse reinforcement;
- \( s \) = spacing of transverse reinforcement along the longitudinal direction of column;
- \( b_c \) = cross-sectional dimension of concrete core perpendicular to the direction under consideration, measured to the center of peripheral transverse reinforcement.

Eqs. (2-43) and (2-45) were derived from Eq. (2-41) with the premise that, if the axial loading capacity of a column can be maintained after the spalling of concrete cover, then the column shall have enough ductility capacity under lateral seismic loading. The Eq. (2-44) and (2-46) provided the lower limits for the large concrete columns with small values of \( (A_g/A_{ch} - 1) \). These equations have been proven to be unsafe in some circumstances, while overly conservative in other situations (Sheikh and Khouy, 1993). Obviously, these design requirements of transverse reinforcement in ACI 318 Code were aimed at achieving certain level of axial deformability for a column under concentric compression, rather than the ductility performance of a column subjected to seismic loading that includes lateral displacement excursions and axial load. Thus, the design requirements have no relationship with the seismic force modification factors.
adopted in the model codes such as IBC-2006 and NBCC-2005. Also notable is that Eq. (2-41) was based on test data of normal-strength concrete and its applicability to high-strength concrete is doubtful.

2.4.3 CAN/CSA-A23.3-04

Confinement design provisions in the Clauses 21.4.4 and 21.7.2 of CAN/CSA-A23.3-04 for concrete columns were based on the performance-based seismic design procedure developed by Paultre and Légeron (2008) with curvature ductility factor of columns as the performance criterion.

Based on nonlinear sectional analysis and parametric studies, Paultre and Légeron (2008) proposed that, for concrete circular and rectangular columns confined by transverse steel reinforcement, the curvature ductility factor $\mu_\phi$ and axial load level $k_p = P/P_o$ is related to the effective confinement index $I_e = f_{le}/f'_c$ as:

$$I_e = 0.0111 k_p \mu_\phi$$  \hspace{1cm} (2-47)

where

- $f'_c$ = compressive strength of unconfined concrete;
- $f_{le}$ = effective confining pressure in concrete by transverse reinforcement at peak stress;
- $P$ = axial load applied on columns;
- $P_o$ = nominal axial load capacity of unconfined column.

The amount of $f_{le}$ is evaluated according to the model of confinement effectiveness developed by Sheikh and Uzumeri (1982) for steel-confined concrete columns, together with the concept of effective stress in transverse reinforcement (Paultre et al., 2001).

Corresponding to the seismic force modification factors specified in NBCC 2005, Paultre and Légeron (2008) categorized the concrete columns in seismic force-resisting frames into two ductility levels as: (1) ductile columns with a ductility-related seismic force modification factor of $R_d = 4.0$ and curvature ductility factor of at least 16, and (2) moderately ductile columns corresponding to a seismic force modification factor of $R_d = 2.5$ and curvature ductility factor of at least 10. It should be noted that in their procedure for the design of confining reinforcement,
Sheikh and Khoury (1997) suggested curvature ductility factors of 16 and 8 for similar categories of columns.

After several simplifications in estimating the effective confining pressure in concrete, Eq. (2-47) was developed into the following simple and conservative seismic design equations, which were adopted in CAN/CSA-A23.3-04 to determine the amount of transverse reinforcement for concrete columns of different ductility categories:

For ductile circle columns with \( \mu_\Phi = 16 \),

\[
\rho_s = 0.40k_p \frac{f'_c}{f_{yh}} \geq 0.45 \left( \frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yh}} \\
\text{(2-48)}
\]

and for moderately ductile circle columns with \( \mu_\Phi = 10 \),

\[
\rho_s = 0.30k_p \frac{f'_c}{f_{yh}} \geq 0.45 \left( \frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yh}} \\
\text{(2-49)}
\]

For ductile square and rectangular columns with \( \mu_\Phi = 16 \),

\[
A_{sh} = 0.20k_p k_n \frac{f'_c}{f_{yh}} \frac{A_g}{A_{ch}} s_{hc} \geq 0.09 \frac{f'_c}{f_{yh}} s_{hc} \\
\text{(2-50)}
\]

and for moderately ductile square and rectangular columns with \( \mu_\Phi = 10 \),

\[
A_{sh} = 0.15k_p k_n \frac{f'_c}{f_{yh}} \frac{A_g}{A_{ch}} s_{hc} \geq 0.09 \frac{f'_c}{f_{yh}} s_{hc} \\
\text{(2-51)}
\]

where \( k_p = P/P_o \) and all the other notations are the same as defined in previous sections.

While the provisions of confinement design in CAN/CSA-A23.3-94 were mainly based on the corresponding provisions in ACI 318 Code, a significant departure was made in the 2004 version of CAN/CSA-A23.3. The confinement requirements are related directly to the performance measured by the curvature ductility factor of columns. The effects of axial load level and reinforcement configuration are also considered.
2.4.4 EN 1998-1:2004

In the Chapter 5 of European Standard EN 1998-1:2004, Eurocode 8: Design of structures for earthquake resistance: General rules, seismic actions and rules for buildings, the requirements for confining steel reinforcement in the potential plastic hinge regions of primary seismic columns were based on a series of semi-empirical expressions of yield and ultimate curvatures, as well as the simplified relationships between the behaviour factor of structures and the local ductility demand of columns. Curvature ductility factor $\mu_\phi$ was taken as the seismic design criterion of the transverse confinement of columns.

As introduced by Fardis (2009), regardless the shape of cross-section, axial load level and reinforcement configuration, the yield curvature $\phi_y$ of concrete columns is predicted approximately by the following semi-empirical expression

$$\phi_y = \frac{\lambda \varepsilon_y}{h_c}$$  \hfill (2-52)

While the ultimate curvature $\phi_u$ of confined concrete column is

$$\phi_u = \frac{\varepsilon_{cu}^*}{\varepsilon_{cu} h_o}$$  \hfill (2-53)

where

- $\lambda$ = empirical factor, taken as 1.75 for column;
- $h_c$ = external depth of cross section of the original unspalled column;
- $h_o$ = depth of cross section of the confined concrete core inside centerline of hoops;
- $\varepsilon_y$ = yield strain of longitudinal reinforcement;
- $\varepsilon_{cu}^*$ = ultimate strain of confined concrete, taken as $\varepsilon_{cu}^* = 0.0035 + 0.2 \frac{p}{f_c}$;
- $p$ = confining pressure in concrete core, estimated by $p = 0.5a \omega_w f_c$;
- $a$ = effective factor of transverse confinement, estimated according to the model proposed by Sheikh and Uzumeri (1982);
- $\omega_w = \rho_w \frac{f_{yw}}{f_c}$, is the volumetric mechanical ratio of confining reinforcement;
\[ \rho_w = \text{volumetric ratio of confining reinforcement to concrete core of column}; \]
\[ f_{yw} = \text{yield strength of confining reinforcement}; \]
\[ f_c = \text{compressive strength of unconfined concrete}; \]
\[ f_c^* = \text{compressive strength of confined concrete, taken as } f_c^* = f_c \left(1 + 3.7 \frac{p}{f_c}\right); \]
\[ \xi_{cu}^* = \text{neutral axis depth of confined concrete core, normalized to } h_o. \]

In order to estimate the neutral axis depth \( \xi_{cu}^* \), a detailed procedure was elaborated by Fardis (2009). After some simplifications, the following expression was derived and adopted by EN 1998-1:2004 to determine the required confinement to the potential plastic hinge region of primary seismic columns

\[ a\omega_{wd} = 30\mu_\Phi v_d \varepsilon_{yd} \frac{b_c}{b_o} - 0.035 \]  

(2-54)

where
\[ \mu_\Phi = \text{required curvature ductility factor of column}; \]
\[ v_d = \frac{N_{Ed}}{A_c f_{cd}}, \text{ normalized design axial force of column}; \]
\[ N_{Ed} = \text{design axial force of column under load combination with earthquake effect}; \]
\[ b_c = \text{external width of cross section of the original unspalled column}; \]
\[ b_o = \text{width of cross section of confined concrete core inside centerline of peripheral hoops}. \]

The subscript \( d \) of several variables in Eq. (2-54) indicates that the design values are used. Additionally, for the primary seismic columns in concrete frames of Ductility Category High (DCH), the minimum value of \( \omega_{wd} \) should be 0.12 within the critical region at the base of the columns (designated potential plastic hinge region). In concrete frames of Ductility Category Medium (DCM), this value should be 0.08.

The most notable respect of the confinement requirement in EN 1998-1:2004 is that the required curvature ductility factor \( \mu_\Phi \) of the potential plastic hinge region of columns is explicitly incorporated in Eq. (2-54), which should be determined according to the corresponding basic value of the behaviour factor \( q_o \) of structures by using the following relationships
\[
\mu_\phi = 2q_o - 1 \quad \text{if } T_1 \geq T_C \quad (2-55)
\]
\[
\mu_\phi = 1 + 2(q_o - 1)\frac{T_C}{T_1} \quad \text{if } T_1 < T_C \quad (2-56)
\]

where

\[T_1 = \text{fundamental period of the building within the considered vertical plane of frame;}\]
\[T_C = \text{period at the upper limit of the constant acceleration region of the spectrum, specified in Clause 3.2.2.2 of EN 1998-1:2004 according to the ground type and type (shape) of spectrum to be used.}\]

The above two expressions are based on the following relationships between the curvature ductility factor \(\mu_\phi\), the local ductility factor \(\mu_\theta\) of the chord rotation at the column ends where plastic hinges form, the global displacement ductility factor \(\mu_\delta\) of the structure and the basic behaviour factor \(q_o\) in the seismic design, which are deemed normally conservative for concrete member by Fardis (2009) and the Clause 5.2.3.4 of EN 1998-1:2004.

\[
\mu_\phi = 2\mu_\theta - 1 \quad (2-57)
\]
\[
\mu_\theta \approx \mu_\delta \quad (2-58)
\]
\[
\mu_\delta = q_o \quad \text{if } T_1 \geq T_C \quad (2-59)
\]
\[
\mu_\delta = 1 + (q_o - 1)\frac{T_C}{T_1} \quad \text{if } T_1 < T_C \quad (2-60)
\]

As a normal engineering practice, for multi-storey and multi-bay regular DCH frames of \(T_1 \geq T_C\), the basic value of the behaviour factor is specified as \(q_o = 4.5 \times 1.3 = 5.85\) in EN 1998-1:2004. Thus, the curvature ductility factor \(\mu_\phi\) used in Eq. (2-54) is \(\mu_\phi = 2 \times 5.85 - 1 = 10.7\) according to Eqs. (2-57) to (2-59). Similarly, for multi-storey and multi-bay regular DCM frames of \(T_1 \geq T_C\), the basic value of the behaviour factor is specified as \(q_o = 3.0 \times 1.3 = 3.9\), so the curvature ductility factor used in Eq. (2-54) is \(\mu_\phi = 2 \times 3.9 - 1 = 6.8\). Note that these targeted curvature ductility factors are much lower than the corresponding design targets taken in CAN/CSA-A23.3-04 and NZS 3101:2006.
Actually, the concept of the chord rotation ductility factor $\mu_\theta$ used by Fardis (2009) is the same as the commonly used displacement ductility factor $\mu_\Delta$ of columns. It is not clear if the simple relationships presented in Eqs. (2-57) and (2-58) are valid for most structures. For example, as demonstrated by Fardis (2009), Eq. (2-57) was developed from the following expression:

$$\mu_\theta = 1 + 3(\mu_\phi - 1) \frac{l_p}{l} \left(1 - 0.5 \frac{l_p}{l}\right) \approx \frac{3l_p}{l} \mu_\phi$$  \hspace{1cm} (2-61)

where $l_p/l$ is the length ratio of the equivalent plastic hinge to the shear span of column. Comparing Eq. (2-57) and (2-61), it is found that Eq. (2-57) is conservative only if $l_p/l \leq 0.184$. Furthermore, normally plastic hinges form sequentially from the lower part of the building and do not extend throughout the intended beam-sway mechanism (Fardis 2009). Thus the maximum value of chord rotation ductility factor $\mu_\theta$ may be about twice of the global displacement ductility factor $\mu_\Delta$ of the whole frame. Similar opinion has also been expressed by Paulay and Priestley (1992) that, for the lower half of the frame, the magnitude of the story displacement ductility factor $\mu_\Delta$ is much larger than the overall displacement ductility factor $\mu_\delta$ for the entire frame and can approximately be taken as $\mu_\Delta = 2\mu_\delta$. Thus, above approach may underestimate the requirement of curvature ductility factor $\mu_\phi$. Lastly, it is found that the factor 30 in the first term of the right hand side of Eq.(2-54) is rounded up arbitrarily from $10 \times 1.75 = 17.5$ in the development of this design equation (Fardis, 2009), which implicitly provides a safety margin on the value of $\mu_\phi$.

### 2.4.5 NZS 3101:2006

Seismic design provisions in Clause 10.4.7 of NZS 3101:2006 to determine the transverse reinforcement in concrete columns were based on the performance-based seismic design procedure developed by Watson et al. (1994), in which the curvature ductility factor of columns was taken as the performance criterion.

Extensive experimental and analytical researches have been conducted in New Zealand to study the seismic behaviour of concrete columns. Based on nonlinear analysis, design charts were proposed by Zahn et al. (1986) to evaluate the demand of transverse reinforcement corresponding to the required curvature ductility factor $\mu_\phi = \phi_\psi/\phi_y$, which were further
developed into the following expressions by Watson et al. (1994).

For circular columns transversely confined by steel spirals or circular hoops, the volumetric ratio of required transverse reinforcement $\rho_s$ is

$$
\rho_s = 1.4 \frac{A_g}{A_c} \left( \frac{\phi_u}{\phi_y} - 33 \rho_t m + 22 \right) \frac{f'_c}{f_{st}} \frac{P}{\rho_{gt} A_g} - 0.0084
$$

(2-62)

and for columns with square and rectangular cross sections, the required rectilinear transverse reinforcement is determined by

$$
\frac{A_{sh}}{s b_c} = \frac{A_g}{A_c} \left( \frac{\phi_u}{\phi_y} - 33 \rho_t m + 22 \right) \frac{f'_c}{f_{yt}} \frac{P}{\rho_{gt} A_g} - 0.006
$$

(2-63)

where

$$
\frac{A_g}{A_c} \geq 1.2 \text{ and } \rho_t m \leq 0.4;
$$

$A_c$ = area of core concrete measured to the outer surface of peripheral transverse reinforcement;

$A_g$ = area of gross cross section of column;

$A_{sh}$ = total effective area of transverse reinforcement in direction of consideration;

$A_{st}$ = total area of longitudinal reinforcement;

$f_{yt}$ = yield strength of transverse reinforcement;

$f_y$ = yield strength of longitudinal reinforcement;

$m = f_y / (0.85 f'_c)$;

$P$ = axial load applied on columns;

$s$ = center-to-center spacing of transverse reinforcement;

$\phi$ = resistance reduction factor, taken as 0.85;

$\phi_y$ = yield curvature;

$\phi_u$ = ultimate curvature;

$\rho_s$ = volumetric ratio of spiral and circular hoop reinforcement to the core of a circular column;

$\rho_t = A_{st} / A_g$, area ratio of longitudinal reinforcement to gross cross section of column.

The requirements of confining reinforcement in Eq. (2-62) and Eq. (2-63) depend on the curvature ductility factor ($\mu_\phi = \phi_u / \phi_y$) as well as the axial load level ($P / (\phi_y A_g)$). The use of $\mu_\phi =$
\( \phi_u / \phi_y = 20 \) and 10 was recommended for ductile and limited ductile columns, respectively, which led to the following design equations adopted by NZS 3101:2006. Additionally, Watson et al. (1994) stated that the transverse reinforcement shall also be sufficient to prevent shear failure of columns and the premature buckling of longitudinal compression bars, which is ensured by the following Eqs. (2-65) and (2-68).

For ductile circular columns with \( \mu \phi \geq 20 \), the required volumetric ratio \( \rho_s \) of spiral or circular hoop reinforcement is the larger of the values given by the following two equations:

\[
\rho_s = \frac{(1.3 - \rho \psi m) A_g f'_c P}{2.4 A_c f_y \phi'_c A_g} - 0.0084
\]  
\[
(2-64)\]

\[
\rho_s = \frac{A_{st} f_y}{110 d''} \frac{1}{f_{yt} d_b}
\]  
\[
(2-65)\]

For limited ductile circular columns with \( \mu \phi \geq 10 \), transverse reinforcement shall be larger of the amounts determined by Eqs. (2-65) and (2-66).

\[
\rho_s = \frac{(1.0 - \rho \psi m) A_g f'_c P}{2.4 A_c f_y \phi'_c A_g} - 0.0084
\]  
\[
(2-66)\]

For ductile square or rectangular columns with \( \mu \phi \geq 20 \), the required total effective area \( A_{sh} \) of transverse reinforcement shall not be less than the larger value determined by the following two equations:

\[
A_{sh} = \frac{(1.3 - \rho \psi m) s_h h'' A_g f'_c P}{3.3 A_c f_y \phi'_c A_g} - 0.006s_h h''
\]  
\[
(2-67)\]

\[
A_{te} = \frac{\sum A_h f_y s_h}{96 f_{yt} d_b}
\]  
\[
(2-68)\]

For limited ductile square or rectangular columns with \( \mu \phi \geq 10 \), transverse reinforcement shall be larger of the amounts determined by Eqs. (2-68) and (2-69).

\[
A_{sh} = \frac{(1.0 - \rho \psi m) s_h h'' A_g f'_c P}{3.3 A_c f_y \phi'_c A_g} - 0.006s_h h''
\]  
\[
(2-69)\]
where all the symbols have been defined before, except that:

- $A_{te}$ = area of one leg of transverse reinforcement;
- $\Sigma A_b$ = sum of area of longitudinal reinforcement supported by the tie under concern;
- $b_c$ = dimension of concrete core, measured to the outer surface transverse reinforcement;
- $d_b$ = diameter of longitudinal reinforcement;
- $h''$ = dimension of concrete core perpendicular to the direction under consideration;
- $s_h$ = center-to-center spacing of transverse reinforcement ($= s$).

Compared to the seismic design provisions for transverse reinforcement in ACI 318-11, the design provisions in NZC 3101:2006 are deemed to be more rational because they relate the confinement requirements directly to the seismic performance measured by the curvature ductility factor of columns. Furthermore, by accounting the influence of axial load, they can lead to safer seismic design for columns under high level of axial load and more economic design for columns subjected to low axial load.

### 2.4.6 CAN/CSA-S806-12

In the Clause 12.5.3 of CAN/CSA-S806-12 (2012), two alternative design approaches are proposed to determine the required thickness of FRP jackets to retrofit concrete columns with deficient transverse confinement, one of which is referred to as the curvature-based approach while the other is a drift-based approach, as introduced in the following sections.

#### 2.4.6.1 Curvature-based approach

This design approach was developed from the following empirical expression (2-70), which was initially proposed by Sheikh and Li (2007) and developed later as part of this study.

For FRP-confined circular and square columns, the relationship between the amount of FRP jackets and the seismic performance, measured by the curvature ductility factor in plastic hinges, was suggested as:

$$t_F \cdot f_{Fu} = \alpha \cdot f'_c \cdot D \cdot Y_p \cdot \frac{\mu_{\phi, in}^{1.15}}{29}$$  \hspace{1cm} (2-70)
where
\[ t_F = \text{total thickness of FRP jackets;} \]
\[ f_{Fu} = \text{ultimate tensile strength of FRP jackets;} \]
\[ \alpha = \text{constant to account for different cross-sectional shapes of columns, taken as}\]
\[ 0.10 \text{ for circular columns and 0.21 for square columns;} \]
\[ f_c' = \text{compressive strength of unconfined concrete;} \]
\[ D = \text{diameter of circular columns or dimension normal to the loading direction of}\]
\[ \text{rectangular columns;} \]
\[ Y_P = 1 + 13 \left( \frac{P}{P_o} \right)^5, \text{factor to reflect the influence of axial load of columns;} \]
\[ P = \text{specified axial load on columns;} \]
\[ P_o = \text{nominal axial load capacity of unconfined columns;} \]
\[ \mu_{\phi,\text{in}} = \mu_{\phi} - \mu_{\phi,\text{org}}, \text{increase in curvature ductility due to transverse FRP-confinement;} \]
\[ \mu_{\phi} = \text{curvature ductility factor of column sections after FRP-retrofit;} \]
\[ \mu_{\phi,\text{org}} = \text{curvature ductility factor of deficient original columns before FRP-retrofit.} \]

From experimental and analytical results investigated in this study, it was found that the curvature ductility factors, \( \mu_{\phi,\text{org}} \), ranged mostly from 2.0 to 4.0 for column sections with negligible transverse confinement. Therefore, the value of \( \mu_{\phi,\text{org}} \) can approximately be taken as 3.0 if a more accurate estimation is not available. The performance criterion in seismic design of FRP-retrofitted columns was taken as follows: the columns in ductile moment-resisting frames should have \( \mu_{\phi} \geq 13 \), while the columns in moderately-ductile moment-resisting frames have \( 8 \leq \mu_{\phi} < 13 \), as proposed by Sheikh and Li (2007). Thereby, the following curvature-based design approach was developed and adopted in CAN/CSA-S806-12 to determine the required total thickness, \( t_F \), of transverse FRP jackets to retrofit deficient concrete columns:

\[ t_F = \beta \cdot Y_P \cdot D \cdot \frac{f_c'}{\phi_F \cdot f_F} \quad (2-71) \]

where,
\[ \beta = \text{a constant to account for different cross-sectional shapes and ductility levels,}\]
\[ \text{taken as: 0.05 for circular columns in ductile moment resisting frames;} \]
\[ \text{0.025 for circular columns in moderately-ductile moment resisting frames;} \]
0.12 for rectangular columns in ductile moment resisting frames;
0.06 for rectangular columns in moderately-ductile moment resisting frames;
\( \phi_F = 0.65 \), material strength reduction factor of FRP jackets;
and all the other parameters were the same as those already defined for the previous equations in this section.

### 2.4.6.2 Drift-based approach

The drift-based approach in CAN/CSA-S806-12 was developed from a displacement-based design approach proposed for steel-confined columns by Saatcioglu and Razvi (2002), in which the lateral drift capacity \( \delta \) was taken as the criterion for seismic performance of columns.

Based on inelastic pushover analysis of concrete columns transversely confined by reinforcing steel, Saatcioglu and Razvi (2002) found that there was a direct correlation among the drift capacity, transverse confinement and the level of axial loading. The following expression was suggested to determine the required volumetric ratio \( \rho_c \) of transverse confining reinforcement in the direction under consideration for columns to achieve a certain drift capacity \( \delta \).

\[
\rho_c = 14 \frac{f'_c}{f_{y,h}} \left( \frac{A_g}{A_c} - 1 \right) \frac{1}{\sqrt{k_c \frac{P_f}{P_{ro}}}} \delta \tag{2-72}
\]

where

- \( P_f/P_{ro} \) = factored axial load level, but should be taken no less than 0.2;
- \( k_c \) = coefficient of confining effectiveness, taken as 1.0 for circular and oval sections, and 0.25 for rectangular and square sections;
- \( A_g/A_c - 1 \) = cross-sectional area ratio of concrete cover to concrete core, but shall be taken no less than 0.3.

For a column transversely confined by externally bonded FRP jackets, the entire cross section of column is wrapped, as opposed to the only core concrete in conventional steel-confined columns. The prior equation was extended to FRP-confined columns by taking \((A_g/A_c) - 1 = 0.3\) by Saatcioglu (2006).

The thickness of the FRP jacket for satisfying a given drift ratio \( \delta \) is determined by
\[ t_F \geq 2D \frac{f_c}{k_c} \frac{P}{P_o} \frac{\delta}{\phi_f f_f} \]  

(2-73)

where

\[ f_{Fj} = \text{workable tensile strength of FRP jackets in the hoop direction of columns;} \]
\[ P/P_o = \text{unfactored axial load level, same as defined for Eq. (2-70); but shall be taken as no less than 0.15;} \]
\[ D = \text{diameter of circular columns or width of rectangular columns, normal to the direction under concern.} \]
\[ k_c = \text{coefficient of confining effectiveness, taken as 1.0 for circular and oval jackets, and 0.4 for rectangular and square jackets;} \]
\[ \delta = \text{design lateral drift ratio, which shall not be less than:} \]
\[ 0.04 \text{ for columns in ductile moment-resisting frames } (R_d = 4.0), \text{ and} \]
\[ 0.025 \text{ for columns in moderately ductile moment-resisting frames } (R_d = 2.5). \]

The workable tensile strength \( f_{Fj} \) in FRP jackets shall be determined according to the cross-sectional shapes of columns, the axial load level, \( P/P_o \), of columns, and the elastic modulus of FRP jacket, \( E_F \), as follows:

1. For columns with circular and oval jackets:
   \[ f_{Fj} = 0.005 E_F \quad \text{when } P/P_o \leq 0.15; \]
   \[ f_{Fj} = 0.01 E_F \quad \text{when } P/P_o \geq 0.30; \]
   with linear interpolation for in-between values of \( P/P_o \).

2. For columns with square and rectangular jackets:
   \[ f_{Fj} = 0.003 E_F \quad \text{when } P/P_o \leq 0.15; \]
   \[ f_{Fj} = 0.006 E_F \quad \text{when } P/P_o \geq 0.30; \]
   with linear interpolation for in-between values of \( P/P_o \).

3. \( f_{Fj} \) shall not be more than \( \phi_f f_{Fu} \).

The drift ratio of columns was taken as the design criterion of seismic performance in this design approach, rather than the curvature ductility factor or displacement ductility factor. This criterion appears problematic since it combined the concepts of ductility and deformability in an irrational way. In engineering practice, the seismic detailing and design provisions are aimed at achieving certain level of ductility factor of structures, corresponding to the adopted seismic
force modification factor $R_d$. Therefore, to determine the required transverse confinement of columns, the suitable design criterion should be ductility factors rather than the drift capacity of columns. This issue will further be discussed in detail later in Chapter 7.

The definition of drift ratio $\delta$ in CAN/CSA-S806-12 is also ambiguous, in which the calculation criterion of drift ratio $\delta$ was not stated explicitly. In the report by Saatcioglu and Razvi (2002), which introduced the background research work for the drift-based design approach in CAN/CSA-S806-12, the drift ratio $\delta$ was defined as the lateral drift capacity of a column corresponding to 20% strength decay. However, it was not clear whether moment or lateral force was represented by the term strength. Only after investigating the proposed expression with the test data in the same report, it was found that $\delta$ was defined here as the lateral drift capacity of a column corresponding to 20% decay of flexural strength. To use this definition, $\delta$ should thus be determined according to the moment at plastic hinge vs. tip deflection ($M-\Delta$) relationship of columns subjected to seismic loading. This relationship is a mix of the sectional and member performance and therefore has rarely been reported in other available literature. The drift ratio defined in CAN/CSA-S806-12 cannot appropriately reflect the $P-\Delta$ effect of column, since the flexural strength of column is just a sectional parameter.

The concept of lateral drift ratio (tagged as $\delta_M$) in CAN/CSA-S806-12 is different from the widely used concept of drift ratio (tagged as $\delta_V$), which is corresponding to 20% strength decay in lateral force resistance and accounts for the $P-\Delta$ effect of columns. A significant portion of moment capacity may be consumed by the $P-\Delta$ effect when the column is enduring large inelastic lateral deflection, therefore the value of $\delta_M$ may be much larger than $\delta_V$. The designers are vulnerable to overestimate the actual drift ratio of columns when determining the amount of FRP confinement using the provisions in CAN/CSA-S806-12. In the commentary to the CAN/CSA-S806-12 guidelines, it was stated that “The minimum design lateral drift ratio was specified ... to be 4%. While this limit is significantly higher than the limit of 2.5% used in the 2005 edition of the National Building Code of Canada, as well as the 2000 edition of the International Building Code of U.S. for new buildings, it may be justified for existing (old) buildings designed prior to the enactment of modern seismic codes which call for appropriate levels of lateral bracing. Furthermore, in retrofitting existing buildings the additional cost of providing extra deformability is often marginal and makes economic sense to go beyond the
minimum level stipulated in the Standard” (Saatcioglu, 2006). Obviously, in the above comparison and commentary, the drift ratio in CAN/CSA-S806-12 was deemed to have the same definition as in NBCC-2005 and IBC-2000. However, it should be noted that limitations of drift ratio in NBCC-2005 is related to the maintenance of lateral force capacity and integrity of structures, while the design equations in this section have been based on maintaining the moment capacity. Comparing the two parameters, $\delta_M$ and $\delta_V$, with different concepts in the commentary points to ambiguity in the design concept.

2.4.7 Sheikh and Khoury (1997), Bayrak and Sheikh (1998) and Sheikh and Li (2007)

A series of systematic performance-based design approaches were developed by Sheikh and Khoury (1997), Bayrak and Sheikh (1998) and Sheikh and Li (2007) to determine the amount of transverse confinement provided by transverse steel reinforcement or FRP-jackets for square and rectangular concrete columns. The effects of axial load level and different confinement efficiency due to reinforcement configuration were taken into account. The proposed approaches were calibrated and corroborated with the experimental results of large-scale square columns tested under simulated seismic loading.

The curvature ductility factor $\mu_\phi$ of the column section was set as the design criterion of seismic performance in these design approaches. For steel-confined columns, the section with $\mu_\phi \geq 16$ was defined as highly ductile, while a moderately ductile section has $8 \leq \mu_\phi < 16$ and the low ductile column has $\mu_\phi < 8$ (Sheikh and Khoury, 1997). Considering the energy dissipation capacity, this criterion was further modified for the FRP-confined columns, in which the section with $\mu_\phi \geq 13$ is defined as high ductile, while a moderately ductile section has $8 \leq \mu_\phi < 13$ and the low ductile column has $\mu_\phi < 8$ (Sheikh and Li, 2007).

For normal-strength concrete columns transversely confined by rectilinear transverse steel reinforcement, the following equation was proposed to relate the amount of transverse reinforcement to the axial load level $P/P_o$ and curvature ductility factor $\mu_\phi$.

$$A_{sh} = \alpha \left\{ 1 + 13 \left( \frac{P}{P_o} \right)^5 \right\} \left( \frac{\mu_\phi}{29} \right)^{15} A_{sh[ACI]}$$

(2-74)
The above design expression was extended by Bayrak and Sheikh (1998) to high-strength concrete tied columns. The following equation was proposed based on the experimental results of steel-confined tied columns with concrete strength ranging between 55 MPa and 115 MPa, in which the brittle characteristic of high strength concrete was displayed clearly.

\[
A_{sh} = \alpha \left\{ 1 + 13 \left( \frac{P}{P_o} \right)^5 \right\} \left( \frac{\mu_s}{8.12} \right)^{0.82} \frac{A_{sh[ACI]}}{
\}
\]

(2-75)

In the above two equations, \( A_{sh[ACI]} \) = transverse reinforcement required by the then current ACI 318 Code, which is the same as ACI 318-11; and \( \alpha \) = constant to take into account the different confinement efficiency due to the reinforcement configuration in columns. The constant \( \alpha \) is 1.0 for columns with tightly knit transverse steel reinforcement in which a minimum of three longitudinal bars are effectively supported by tie corners or hooks on each column face. However, when tied columns have only single peripheral hoop as confining steel, \( \alpha \) is taken as 2.5 for normal strength concrete columns and 1.35 for high-strength concrete columns.

Based on the more recent new experimental results, Eq. (2-74) is further extended to circular columns as follows:

\[
\rho_s = \left\{ 1 + 13 \left( \frac{P}{P_o} \right)^5 \right\} \left( \frac{\mu_s}{7} \right)^{0.7} \rho_s[ACI] \]

(2-76)

where \( \rho_s \) = required volumetric ratio of transverse reinforcement in circular columns; and \( \rho_s[ACI] \) = volumetric ratio of transverse reinforcement to concrete core of column, required by ACI 318-11 for circular columns.

On the lines of the prior approaches for steel confined columns, Sheikh and Li (2007) developed a procedure to determine the required external FRP jackets in the FRP-retrofitted square columns. In this procedure, the amount of FRP confinement is related to the level of axial load and the ductility performance in terms of curvature ductility factor as follows.

\[
n \cdot f_{ru} = \beta \cdot h \cdot f' \cdot \left\{ 1 + 13 \left( \frac{P}{P_o} \right)^5 \right\} \frac{\mu_{\phi80, in}}{29}^{1.15}
\]

(2-77)

where

\[
\mu_{\phi80, in} = \mu_{\phi80} - \mu_{\phi80, org}, \quad \text{increase in curvature ductility due to FRP confinement;}
\]
\( n \) = number of layers of FRP jackets;

\( f_{Fu} \) = ultimate tensile strength of each layer FRP jacket, obtained from tensile
coupon tests (N/mm/per layer);

\( f'_c \) = compressive strength of unconfined concrete (MPa);

\( \beta \) = constant, taken as 0.25 for square columns;

\( h \) = cross-sectional dimension of column perpendicular to the direction under
consideration;

\( P \) = axial load applied on columns;

\( P_o = 0.85f'_c (A_g - A_{st}) + f_A A_{st} \), the nominal axial load capacity of column as defined in
ACI 318-11;

\( \mu_{\phi_{80}} \) = curvature ductility factor of columns after FRP-retrofit;

\( \mu_{\phi_{80,org}} \) = curvature ductility factor of the original columns without FRP-retrofit.

The above procedure was initially calibrated for the FRP-confined square columns, but it can
also be extended to FRP-retrofitted circular columns by taking into account the difference in
confinement efficiency. For similar ductility enhancements, about half of the amount of FRP
wrapping is required to retrofit circular columns compared to equivalent square columns.

### 2.4.8 Comparison of Design Methods

The confinement requirements in several current design codes (ACI 318-11; CAN/CSA-A23.3-
04; EN 1998-1:2004; NZS 3101:2006; CAN/CSA-S806-12) and the performance-based design
procedures presented above (Sheikh and Khoury, 1997; Bayrak and Sheikh, 1998) are compared
in this section.

For steel-confined columns in ductile moment-resisting frames (or DCH frames in EN 1998-1),
the required amounts of confining reinforcement by current design codes (ACI 318; CAN/CSA-
A23.3; EN 1998-1; and NZS 3101) and the performance-based design procedure are estimated
for two example columns. As mentioned previously, only the confinement requirements are
studied in the examples while the details of anti-buckling requirement and shear resistance of
columns are not considered. One example is a circular column of 500 mm diameter and another
is a square column with 500×500 mm cross section. In both columns the specified concrete
cylinder strength is \( f'_c = 30 \) MPa; eight 25 mm diameter Grade 400 steel bars are used as
longitudinal reinforcement, while 10 mm diameter Grade 400 steel is used as transverse reinforcement with 40 mm clear concrete cover measured to the outer surface of transverse reinforcement. All strength reduction factors are taken as unity. For EN 1998-1:2004, the specified prism strength of concrete is taken as \( f_c = 0.85f'_c = 0.85 \times 30 = 25.5 \) MPa and the confinement effectiveness factor \( a \) in this code is approximately taken as 0.9 for the circular example column and 0.56 for the square column. The comparison of required amount of confining steel reinforcement is illustrated in Figure 2.7. This figure clearly demonstrates the large variation in the amount of confining reinforcement required by various design codes and performance-based design procedures. Except ACI 318-11, all the design approaches considered the influence of axial load on the curvature ductility performance of columns. For the example columns, the amount of confinement required by EN 1998-1:2004 is the highest in most situations. As a result of ignoring the influence of axial load, the requirement of ACI 318-11 is higher than the requirements in other codes when the columns are subjected to low axial load, while is on the deficient side if the columns are subjected to high axial load.

In order to illustrate the design of FRP-retrofit to columns initially deficient in transverse confinement, the amounts of transverse FRP-confinement required for the preceding two example columns are determined by the curvature-based and the drift-based methods of CAN/CSA-S806-12. The transverse confinement is provided solely by FRP jackets without any transverse steel reinforcement to retrofit these two columns to ductile or moderately-ductile levels. The ultimate tensile strength and rupture strain of the FRP jackets is taken as 1500 MPa and 0.02, respectively, while the nominal FRP thickness is 0.35 mm per layer. The material strength reduction factor of FRP jackets is taken as 1.0. As shown in Figure 2.8, both approaches in CAN/CSA-S806-12 reflect similar trend of the influence of axial load on the requirement of FRP-confinement. In order to retrofit the circular example column to ductile or moderately-ductile level, both approaches lead to similar design results. However, for the square column, the required FRP-confinement by the drift-based approach is slightly more than that by the curvature-based approach. Besides the different theoretical background behind each approach, this difference is mainly caused by the lower limit for the effective FRP strain in the drift-based approach, especially in the cases of square columns. As reported in many seismic tests of columns, the measured tension strain in FRP jackets can develop up to more than 1.5% and approaches the ultimate tensile strength of the FRP composites (Ozbakaloglu and
Saatcioglu, 2006). This limitation is even more conservative for GFRP jackets which may have an ultimate strain of more than 2.0%.

(a) Circular Column

(b) Square Column

Figure 2.7 Steel confinement design examples by different codes and proposals
Figure 2.8 FRP confinement design examples by CAN/CSA S806-12
2.4.9 Summary

The relevant studies reported in scientific literature are summarized in this chapter. Different analytical models and design approaches are discussed, while the provisions for the design of transverse confining steel and FRP reinforcement are critically examined. Application of different design procedures to columns with circular and square sections reveal a large variation in the design requirements. The amount of required FRP jackets using the two alternative methods of CAN/CSA-S806-12 seem to provide the closest agreement among available procedures. Except for CAN/CSA-S806-12, detailed design procedures to determine the required FRP jackets for FRP-retrofitted columns are still not available in other design codes and guidelines, such as CAN/CSA-S6-06 and ACI 440.2R-08.

In the seismic design provisions of ACI 318-11, the ductility of columns is not considered directly while the design criterion for confining reinforcement is the maintenance of axial load capacity after the spalling of concrete cover. Most other design codes include the effect of axial load levels on the confining requirement. Results from an example show that the amount of transverse steel required by one code can be as high as 3 times that required by another code.

The large variations in the design of transverse steel confinement and the lack of design guidelines for the FRP confinement in most codes point to a need for a comprehensive investigation of the subject.
Chapter 3
Experimental Program

3.1 General

In this experimental program, fifteen large-scale circular concrete columns with different types and amount of transverse confinement were tested under constant axial compression and laterally cyclic quasi-static loading simulating earthquake. The main variables were axial load level, spacing of spirals, type of confinement (steel vs. FRP), and type and amount of FRP jackets. In the following sections, the material properties, configuration and construction of specimens, instrumentation and test setup are described in detail.

3.2 Materials

3.2.1 Concrete

To minimize the variation of concrete between samples, all fifteen specimens were cast in one batch of normal-weight concrete with a specified 28-day compressive strength of 30 MPa. Type 10 Portland cement and 10 mm maximum aggregate size were specified for the concrete mixture, with a 150 mm slump at the time of casting.

The development of concrete strength with age was monitored by tests (ASTM C39 Standard) of 150×300 mm concrete cylinders that were cured adjacent to the column specimens under similar conditions. The load was applied at a constant rate of 4.4 kN/sec until the cylinders failed due to concrete crushing. This loading rate is equivalent to 0.24 MPa/sec, falling within the ASTM requirement range of 0.14 - 0.34 MPa/sec. There were at least three cylinders in each group which were tested at 7, 14, and 28 days after casting, and throughout the columns testing period. Testing of column specimens started at 91 days after casting of concrete and completed at 150 days, during which the strength of concrete $f'_c$ did not change much and was measured between 39.6 and 40.5 MPa. Hence, $f'_c$ was taken as 40 MPa for all columns. The compressive strength of concrete developing with age is shown in Figure 3.1, in which each point represents the average of at least three cylinder tests.
### 3.2.2 Steel reinforcement

Three types of deformed steel bars were used in the specimens. Grade 400 25M bars were used as longitudinal reinforcement, while Grade 400 10M and Grade 60 US#3 steels were used as transverse reinforcement. Their mean values of mechanical properties were determined from tension tests using an MTS test system in accordance with ASTM A370-03a Standard using a minimum of three samples of each type of steel and are presented in Table 3.1, while their stress-strain curves under tension are shown in Figure 3.2.
Table 3.1 Mechanical properties of reinforcing steels

<table>
<thead>
<tr>
<th>Rebar type</th>
<th>Area $A_s$ (mm$^2$)</th>
<th>Yield strength, $f_y$ (MPa)</th>
<th>Yield strain, $\varepsilon_y$</th>
<th>Elastic modulus, $E_s$ (MPa)</th>
<th>Start of Strain hardening, $\varepsilon_{sh}$</th>
<th>Ultimate strength, $f_u$ (MPa)</th>
<th>Strain at strength $f_u$, $\varepsilon_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>10M</td>
<td>100</td>
<td>450</td>
<td>0.0024</td>
<td>191431</td>
<td>0.0213</td>
<td>583</td>
<td>0.2123</td>
</tr>
<tr>
<td>25M</td>
<td>500</td>
<td>490</td>
<td>0.0024</td>
<td>201136</td>
<td>0.0250</td>
<td>641</td>
<td>0.1956</td>
</tr>
<tr>
<td>US#3</td>
<td>71.3</td>
<td>496</td>
<td>0.0025</td>
<td>198580</td>
<td>0.0283</td>
<td>605</td>
<td>0.1711</td>
</tr>
</tbody>
</table>

3.2.3 Fiber reinforced polymers

To retrofit the seven concrete columns with inadequate steel confinement, two types of Tyfo® Composite FIBERWRAP Systems were used, i.e. Tyfo® SEH-51S Glass Fiber Reinforced Polymer (GFRP) and Tyfo® SCH-41S Carbon Fiber Reinforced Polymer (CFRP). The FRP composites were made by impregnating unidirectional fibers with Tyfo®-S Epoxy adhesive and curing until hardening.

The FRP coupons were tested according to the ASTM D3039 Standard to measure the mechanical properties of the FRP materials. As shown in Figure 3.3, each 175 mm long FRP coupon comprised one 25×175 mm center piece with the fiber orientated in the long direction and two 25×50 mm reinforcing pieces at each end to protect the center piece under applied high clamping pressure. The actual thickness of each FRP layer depended on the amount of epoxy and was not strictly controlled. Since the mechanical properties of the FRP composites did not change appreciably by the amount of epoxy, the tensile strength of the FRP was determined by using its nominal thickness.

During testing, the two ends of the FRP coupon were clamped firmly by metal grips and tested under direct tension until the FRP rupture. The tensile strain of the FRP was measured by 20 mm electronic strain gauges mounted at the center of the FRP coupon, along the direction of the fibers. The tensile stress-strain relationships of all FRP coupons were almost linearly elastic until rupture as shown in Figure 3.4. At least five coupons were tested for each FRP composites and the average values of mechanical properties are presented in Table 3.2. As a reference, the mechanical properties reported by the manufacturer of the dry fabric and epoxy resin materials are also presented in the table.
FRP composites | Nominal thickness $t_{fr}$, mm | Tensile strength $f_{fu}$, MPa | Rupture strain, $\varepsilon_{fu}$ | Elastic modulus $E_f$, MPa
---|---|---|---|---
GFRP coupon | 1.25 | 518 | 0.02031 | 25488
CFRP coupon | 1.00 | 939 | 0.01229 | 76433
Dry Glass fabric* | 0.36 | 3240 | 0.045 | 72400
Dry Carbon fabric* | 0.37 | 3790 | 0.017 | 230000
Epoxy resin* | - | 72.4 | 0.050 | 31800

*Reported by manufacturer.
3.3 Test specimens

Each specimen consisted of a 356 mm diameter and 1473 mm long concrete column cast monolithically with a 508×762×813 mm reinforced concrete stub, as shown in Figure 3.5. The column part represented a portion of a bridge column or a building column between the section of maximum moment and the point of contraflexure, while the stub represented a discontinuity such as a beam-column joint or a footing.

![Figure 3.5 Details of test specimen](image)

The specimens were classified into two categories according to the material used for transverse confinement. The first category of specimens were referred to as steel-confined columns and consisted of eight columns which were solely reinforced by conventional steel cages of longitudinal and spiral reinforcement. The second category of specimens, referred to as FRP-confined or FRP-retrofitted columns, included seven columns which initially contained only minimal amount of spiral steel (US#3@300 mm) and were later retrofitted with transverse GFRP or CFRP wrapping. The longitudinal steel reinforcement in both types of columns was similar. The details of each specimen are presented in Table 3.3. The volumetric ratio of transverse steel reinforcement to concrete core, $\rho_s$, in the potential plastic-hinge region is listed for each column and the amount of spiral reinforcement relative to the requirements of various design codes is also given in the table. The amount of $\rho_s(ACI)$ is required by ACI 318-11, while $\rho_s(CSA)_1$ and $\rho_s(CSA)_2$ are the requirements by CAN/CSA-A23.3-04 for columns in ductile and
Table 3.3 Details of test specimens

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen</th>
<th>$f'_c$ (MPa)</th>
<th>Transverse reinforcement</th>
<th>Axial load level</th>
<th>FRP-retrofit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Size/@spacing (mm)</td>
<td>$\rho_s$ (%)</td>
<td>$f'_yh$ (MPa)</td>
</tr>
<tr>
<td>1</td>
<td>P27-NF-1</td>
<td>40</td>
<td>US#3@150</td>
<td>0.60</td>
<td>496</td>
</tr>
<tr>
<td>2</td>
<td>P27-NF-2</td>
<td>40</td>
<td>US#3@100</td>
<td>0.90</td>
<td>496</td>
</tr>
<tr>
<td>3</td>
<td>P27-1CF-3</td>
<td>40</td>
<td>US#3@300</td>
<td>0.30</td>
<td>496</td>
</tr>
<tr>
<td>4</td>
<td>P27-2GF-4</td>
<td>40</td>
<td>US#3@300</td>
<td>0.30</td>
<td>496</td>
</tr>
<tr>
<td>5</td>
<td>P40-NF-5</td>
<td>40</td>
<td>US#3@300</td>
<td>0.30</td>
<td>496</td>
</tr>
<tr>
<td>6</td>
<td>P40-NF-6</td>
<td>40</td>
<td>US#3@100</td>
<td>0.90</td>
<td>496</td>
</tr>
<tr>
<td>7</td>
<td>P40-NF-7</td>
<td>40</td>
<td>US#3@75</td>
<td>1.20</td>
<td>496</td>
</tr>
<tr>
<td>8</td>
<td>P40-1CF-8</td>
<td>40</td>
<td>US#3@300</td>
<td>0.30</td>
<td>496</td>
</tr>
<tr>
<td>9</td>
<td>P40-1GF-9</td>
<td>40</td>
<td>US#3@300</td>
<td>0.30</td>
<td>496</td>
</tr>
<tr>
<td>10</td>
<td>P56-NF-10</td>
<td>40</td>
<td>US#3@300</td>
<td>0.30</td>
<td>496</td>
</tr>
<tr>
<td>11</td>
<td>P56-NF-11</td>
<td>40</td>
<td>10M@100</td>
<td>1.22</td>
<td>450</td>
</tr>
<tr>
<td>12</td>
<td>P56-NF-12</td>
<td>40</td>
<td>10M@75</td>
<td>1.63</td>
<td>450</td>
</tr>
<tr>
<td>13</td>
<td>P56-2CF-13</td>
<td>40</td>
<td>US#3@300</td>
<td>0.30</td>
<td>496</td>
</tr>
<tr>
<td>14</td>
<td>P56-3GF-14</td>
<td>40</td>
<td>US#3@300</td>
<td>0.30</td>
<td>496</td>
</tr>
<tr>
<td>15</td>
<td>P56-4GF-15</td>
<td>40</td>
<td>US#3@300</td>
<td>0.30</td>
<td>496</td>
</tr>
</tbody>
</table>

Notes:  
$f'_c$ = compressive strength of concrete, MPa  
$P$ = applied constant axial load, kN  
$P_o$ = theoretical axial load capacity according to ACI 318-11, kN  
$s$ = spacing of transverse spiral steel, mm  
$\rho_s$ = volumetric ratio of transverse spiral steel to concrete core, %
moderately-ductile concrete frames, respectively. The amount of $\rho_{s(NZS)1}$ and $\rho_{s(NZS)2}$ are corresponding to the requirements of NZS 3101:2005 for ductile and limited ductile columns, respectively.

All the fifteen specimens can also be categorized into three groups according to the constant axial load sustained by them during testing. Axial load on four columns was $0.27P_o$ (i.e. 1283 kN), while five were tested under $0.40P_o$ (i.e. 1901 kN) and the remained six were subjected to $0.56P_o$ (i.e. 2661 kN), where $P_o = 0.85f'_c(A_g - A_{st}) + f_yA_{st} = 4752$ kN, which is the nominal concentric axial load capacity of columns according to ACI 318-11.

Annotation of the specimen labels in Table 3.3 are as follows:

- $P_{xx}$ = level of applied axial load, as $xx\%$ of $P_o$
- NF = non-retrofitted by FRP jackets;
- $n\text{CF}$ = retrofitted by $n$ layers of CFRP jackets;
- $n\text{GF}$ = retrofitted by $n$ layers of GFRP jackets.

The number (-1 to -15) at the end of the specimen label indicates the sequence of testing.

### 3.4 Construction of specimens

#### 3.4.1 Reinforcing cages

Each column contained six 25M longitudinal reinforcing bars, resulting in a volumetric ratio of 3.01%. As illustrated in Figure 3.5, the longitudinal steel bars were arranged evenly around the concrete core of the column and embedded to near the bottom of footing stub.

The transverse reinforcement consisted of US#3 spirals, except in specimens P56-NF-11 and P56-NF-12 in which the spirals were made of 10M rebars. The spacing of spirals within the test region of column is shown in Table 3.3 for each specimen. The test region of column is defined here as 800 mm long adjacent to the stub-column interface. To ensure that failure occurred within the potential plastic-hinge region of columns, the spacing of spirals outside the test region was reduced to two-third of the specified spacing within the test region. Spirals were extended into stubs for 100 mm and both their ends were bent around longitudinal reinforcement inside the core. The clear concrete cover was 20 mm to the outer surface of spirals in all the columns,
which led to the area ratio of the concrete core to the gross cross section of column of about 74%, measured to the centerline of spirals.

In order to avoid failure in the stub during the testing, its steel cage was designed with 10M@64 mm stirrups horizontally and vertically.

### 3.4.2 Formwork

The formwork of each specimen consisted of two parts: the base form for the stub and the round Sonotube for the column. The base form was constructed with 19 mm plywood and 38×89 mm wood studs. Sixteen steel supporters were installed externally around the base form to provide extra lateral support to prevent any significant transverse expansion during concrete casting.

All the drilled screw holes in the forms were filled with wood filler after assembling of the formwork to ensure an even concrete surface. To avoid bond between the concrete and formwork and also to eliminate the water absorption by forms, all the inner surfaces of the formwork were coated with a layer of varnish. Furthermore, before setting the reinforcing cages into the base form, the inner form-surfaces were lightly coated with a thin layer of form-release oil.

After setting the reinforcing cages into the base form, Sonotubes were slid down along fifteen steel cages of columns to the top face of the stubs. After the Sonotubes were plumbed to ensure that all the columns were vertically straight and lined up, they were fastened into place by two strong wood frames at both ends. The bottom frame was fixed directly upon the top of the base form, while the top frame was supported to the base form using diagonal SPF studs. The steel cages and the finished formwork are shown in Figure 3.6.

Some additional measures were taken to facilitate the testing. Before the casting of concrete, ten internally threaded rods of 10 mm diameter were installed within the test region of each specimen through holes drilled on the sides of the Sonotube. These rods would be used later to hold LVDT mounts to measure the deformation of concrete core during testing. The corners of the formwork and the holes around the threaded rods were sealed using silicone caulking. In order to install the specimen into the test frame, six 5/8 in. threaded anchors were preinstalled at the center of each end of the specimen. Thus, each specimen had six prefabricated steel anchors.
fixed into its base formwork and another six screwed onto a piece of circular plywood of 356 mm diameter, which were installed at the top of each column at the end of concrete casting.

![Steel-cages and finished formwork for specimens](image)

**Figure 3.6 Steel-cages and finished formwork for specimens**

### 3.4.3 Concrete casting and curing

All the specimens were cast using the same batch of ready-mixed concrete. The slump of the concrete was initially 100 mm and increased to 150 mm by adding super-plasticizer. The fifteen
stubs were cast first and then the columns. All the specimens were appropriately vibrated using rod vibrators. Thirty 152×305 mm concrete cylinders were also made at the same time for the monitoring of concrete strength.

At the end of casting, six anchors pre-screwed onto a piece of circular plywood were forced into the top of each column with appropriate orientation. After casting finished, the specimens and cylinders were covered by wet burlap and polyethylene sheets to provide moist curing environment. On the seventh day after casting, the formworks of all the specimens and cylinders were removed. They were then air-cured in the laboratory under identical conditions until testing. The laboratory temperature was around 20°C with a relative humidity of 40% to 60% during the curing period.

3.4.4 FRP wrapping

Seven specimens were transversely confined by the GFRP or CFRP jackets before testing, using the following procedure. Firstly, the Tyfo®-S Epoxy for the Tyfo® Composite FIBERWRAP System was prepared. The epoxy was composed of two components named A and B, mixed in a volumetric ratio of 100 to 42. Prior to being wrapped around the columns, the unidirectional glass or carbon fabric was saturated with epoxy on both sides using roller brushes. The column surface was also coated with epoxy to improve the concrete-fabric bond. The saturated fabric was then hand-wrapped around the columns with fibers aligned in the circumferential direction of columns. Care was taken to ensure that each composite layer was tightly wrapped without entrapped air bubbles or distortion of fabric. Excess epoxy was squeezed out.

As no strength contribution in the longitudinal direction of the column was expected from the FRP jackets, they were not embedded in the footing (stub) and a gap of 5 mm was maintained between the footing top and FRP jackets. The overlap length at the end of FRP wraps was 150 mm in the circumferential direction for bond integrity. Along the locations where the FRP jackets were punctured through by the threaded rods for mounting LVDTs, one more layer of 600×75mm FRP strip was installed with the 75mm long fiber in the circumferential direction to prevent premature failure at these weakened locations.

Similar to the steel-confined specimens, the design of the FRP-confined specimens also aimed at forcing the column failure to occur in the potential plastic hinge region, i.e. within the 800 mm
long test region adjacent to the column-stub interface. Therefore outside the 800 mm long test region, the columns were wrapped with one more layer of FRP than the specified amount for plastic hinge regions listed in Table 3.3. Moreover, in the three CFRP-confined columns, the CFRP jackets outside test regions were replaced by twice the number of layers of GFRP wraps to reduce cost. Lastly, for the three conventional steel-confined specimens under high axial load of $0.56P_o$, four layers of GFRP jackets were wrapped around the columns outside the test regions to prevent failure there.

According to the product specification from the manufacturer, approximately 90% of the epoxy strength is gained in the first twenty-four hours. The epoxy was cured for at least six days to gain full strength before testing.

### 3.5 Instrumentation

Extensive instrumentation was utilized to collect the test data during each experiment, which includes the axial and lateral loads, the deflections along the specimen length, the strains in concrete, steel reinforcement and FRP jackets. Additionally, visible observations were recorded with the help of still cameras and video cameras during the whole test process.

#### 3.5.1 Strain gauges

Electrical resistance strain gauges were installed to measure the strains in steel reinforcement and the FRP jackets during testing. Two kinds of electric strain gauges were used for different purposes. Twenty-four 2 mm strain gauges (FLT-2-11-5LT) were mounted on the steel reinforcement in each specimen, while additional eight 60 mm strain gauges (FLT-60-11-5LT) were used on the FRP jacket of each FRP-retrofitted column.

In each specimen, eighteen strain gauges were installed on the surface of longitudinal reinforcement, while the strain in spirals within the potential plastic-hinge region was measured by six strain gauges (three on each of the two turns adjacent to the stub face), as shown schematically in Figure 3.7. The strain gauges were mounted on the neutral axis of steel bars in the case of possible bending and buckling, so that the effect of steel bending and buckling could be minimized as much as possible. The general procedure for installing strain gauges on steel reinforcement was as follows: First, the ribs of the deformed bars were ground down and the
surface was smoothened by first using a handhold pneumatic grinder and then by water and ultra-fine sand paper. After a reasonably smooth surface was achieved, it was cleaned by a water-based acidic surface cleanser and an alkaline neutralizer (M-Prep). Then, strain gauges were stuck onto the steel surface by strain gauge adhesive (Cyanoacrylate Glue) and covered with tape. The tape was removed after several minutes and two layers of air-drying polyurethane coating material (M Coat A) were painted onto the strain gauges for waterproofing. Wires of 4.0 m length were pre-soldered to the gauge terminals to connect strain gauges to the data acquisition system. Finally, the strain gauges and their terminals were fully covered with a wax layer and a self-adhesive aluminum foil, after the wires were labeled and taped to the rebars.

For each of the seven FRP-retrofitted columns, eight 60 mm strain gauges were mounted on the surface of the FRP jackets to monitor the strain development in the fiber direction. Figure 3.8 shows the location of eight strain gauges around the column circumference, four at 150 mm and other four at 300 mm from the column-stub interface. Since the FRP surface was not sufficiently even for mounting strain gauges, a thin layer of PS-paste patch was spread on the cleaned FRP surface prior to sticking strain gauges.

![Diagram](image-url)
(All dimensions in $mm$)

Figure 3.8 Locations of strain gauges on FRP surface

Figure 3.9 LVDTs arrangement
3.5.2 Linear variable differential transformers (LVDT)

Twenty-four LVDTs were employed in each specimen, as shown in Figure 3.9. The longitudinal and diagonal deformations in the concrete core within the potential plastic hinge region were measured using eighteen LVDTs, with ten on the north side and eight on the south side of the specimen. The gauge lengths varied from 75 to 300 mm and covered a column length of about 475 mm. These LVDTs were mounted between the threaded rods, which were installed in specimens before the concrete casting. This allowed measurement of the deformation of concrete core even after the spalling of concrete cover of columns. Two LVDTs on the south side of the specimen were installed in the diagonal direction to measure shear deformations. Lateral deflection of the specimen along the longitudinal length was measured by six LVDTs, which were mounted vertically and firmly on the floor.

3.6 Testing

3.6.1 Test setup

The experiments were conducted using a specially designed column test frame at the University of Toronto. The column was tested horizontally and subjected simultaneously to a constant axial load and cyclic quasi-static lateral loading, as shown by the test setup in Figure 3.10. The lateral load was applied at the stub, so that the most critically loaded region of the column was adjacent to the stub and subjected to combined flexure, shear and axial loading.

The axial load was applied by a hydraulic jack with 4450 kN capacity, while the laterally cyclic loading was applied by an MTS actuator with 1000 kN loading capacity and ±100 mm stroke capacity. Two specially designed steel hinges allowed each end of specimen to rotate in plane and kept the axial load path constant throughout testing. In order to install the specimen in the test frame, a 64 mm thick steel plate was bolted to each end of the specimen utilizing six 5/8 in. steel anchors that were embedded during construction. The excess gap in the holes in the plates provided enough flexibility to centrally align the plates and the specimen with ease. The specimen was then lifted up and connected to the hinges in the frame using high strength bolts. The rotation axes of the hinges were the centroidal axes of their shafts, which were used to support the specimen inside the test frame. This design of the test setup allowed the specimen to
rotate freely at the ends with minimal friction even under very high axial load; therefore the mechanism of axial load application remained undisturbed throughout testing.

It should be noted that the length of the concrete columns was 1473 mm but the test setup resulted in an actual shear span of 1841 mm measured from the centoidal axis of the right steel shaft to the column-stub interface (i.e. from the contraflexure point to the column section with maximum moment). Therefore, the shear span-depth ratio of the column was 5.17, where the depth was defined as the outside diameter of column.

Figure 3.10 Test setup

3.6.2 Testing procedure

Each specimen was strictly aligned before testing, so that the center line of specimens coincided with the action line of axial load. Engineering levels and plumb bobs were used to initially align each specimen in both vertical and horizontal planes. Then, the axial load was applied to the specimen in 100 kN increments up to 50% of the specified axial load. In each incremental step the deformations on four sides of the column were monitored by LVDTs over a gauge length of
475 mm from the stub face. If the difference between the average and the maximum or minimum deformation was more than 5%, the specimen was unloaded, adjusted and reloaded, until this 5% criterion was met. After the specimen was finally positioned, the axial load was reduced to about 5 to 10 kN and all instruments were initialized.

At the beginning of each test, the predetermined axial load was firstly applied to the specimen and kept constant throughout the test by adjusting a manually-controlled load maintainer. After the application of the axial load, the specimen was connected to the 1000 kN MTS actuator by clamping two sets of steel plates on the top and bottom of the stub using four 32 mm diameter high strength all threaded rods. Figure 3.11 shows the frame with a column specimen in place.

![Figure 3.11 Test frame with specimen in place](image)

The cyclic lateral loading was applied in a displacement-controlled mode according to the specified lateral deflection history shown in Figure 3.12. The lateral loading sequence consisted of one cycle to a peak deflection of $0.75\Delta_y$, followed by two cycles each one to $\Delta_y$, $2\Delta_y$, $3\Delta_y$...
and so on, until the column was unable to support the applied axial load. As illustrated in Figure 3.12, the nominal yield deflection $\Delta_y$ was defined corresponding to the nominal lateral load capacity $V_n$ along a straight line joining the origin and a point of $65\% V_n$ on the ascending branch of the lateral shear vs. tip deflection curve. To evaluate the amount of $V_n$, elastic-perfectly plastic stress-strain relationship was assumed for longitudinal steel while strain-hardening and buckling of steel were ignored. Meanwhile, the stress-strain relationship of unconfined concrete was used with the ultimate strain of 0.0035. All resistance reduction factors were taken as unity. The $V_n$ vs. $\Delta$ response was determined with specific axial load applied on the column while the $P$-$\Delta$ effect was taken into account.

All the test data was collected automatically at specified intervals using a Hewlett Packard data acquisition system and stored in a computer.

Figure 3.12 Specified lateral deflection history
Chapter 4
Experimental Results

4.1 General

Experimental observations and test results of the fifteen specimens are reported in this chapter. Test results are presented for each column in the form of hysteresis loops of the shear force vs. tip deflection ($V-\Delta$) relationship, the applied lateral load vs. displacement ($P_L-\delta_L$) relationship, the moment vs. curvature ($M-\phi$) relationship at the critical sections and the moment vs. tip deflection ($M-\Delta$) relationship, together with various ductility parameters. Also discussed are the topics about the effects of axial load level, type and amount of transverse confinement, in addition to flexural strength enhancement caused by transverse confinement.

As part of this comprehensive research program, nine circular columns with steel- or FRP-confinement were tested by Sheikh and Yau (2002) which were introduced in Chapter 2. These tests were conducted in the same manner which allows a direct comparison. Therefore, the results of these tests are also included in the following analysis.

4.2 Test observation

All the fifteen specimens in this experimental program are classified into two categories according to the type of transverse confinement. In the first category, eight steel-confined concrete columns were reinforced with conventional longitudinal steel and spirals. The second category consisted of seven FRP-confined concrete columns which initially contained only minimal spirals and then strengthened with transverse CFRP or GFRP jackets. The distinct behaviour of the two categories of specimens is presented individually in the following sections.

4.2.1 Steel-confined columns

Two specimens in this category (i.e. P40-NF-5 and P56-NF-10) had only #3@300 mm spirals in the test regions. The effect of transverse confinement by this kind of widely spaced spirals is very weak and can be considered negligible (Sheikh and Uzumeri, 1982). These two columns represented the deficient columns designed typically in accordance with pre-1970s concrete design codes and were utilized here as unconfined columns or the so-called control specimens.
The other six specimens in this category (i.e. P27-NF-1, P27-NF-2, P40-NF-6, P40-NF-7, P56-NF-11 and P40-NF-12) were transversely confined by different amount of closely spaced spirals. They were designed as columns in ductile or moderately-ductile moment-resisting concrete frames according to CAN/CSA-A23.3-04.

The two control specimens behaved very similarly under laterally cyclic loads, even though they were subjected to different levels of axial loading. At the end of the 1<sup>st</sup> cycle of lateral displacement excursion, very tiny flexural cracks could be observed on the top and bottom surfaces in the test region of columns over a range of about 600 mm. Starting in the 2<sup>nd</sup> lateral cycle in which the peak deflection is $\Delta_y$, the initiation of concrete cover spalling was observed within the maximum moment zone of columns. In the 4<sup>th</sup> cycle with a peak deflection of $2\Delta_y$, the spalling of concrete cover developed very quickly with the increase of lateral deflection, and the axial compressive resistance could not be maintained anymore. The columns collapsed with the crushing of concrete core. No spiral fractured while longitudinal rebars buckled outwards severely between two adjacent spirals in these two control specimens. Figures in Appendix A show the failed zones of the specimens.

The damage of the other six steel-confined columns also occurred within their test regions. Tiny flexural cracks appeared on the top and bottom surfaces of the columns during the 1<sup>st</sup> cycle of lateral load. Additional cracks emerged and existing cracks widened gradually as the lateral deflection increased in the following two cycles. The vertical flexural cracks formed during the first three cycles were mainly concentrated within a range of 300 to 400 mm of columns from the column-stub interface, and occurred much closer to the interface during the following cycles. These flexural cracks developed into flexural-shear cracks as they propagated into the column sides. The initiation of concrete cover spalling could be observed after the 3<sup>rd</sup> lateral cycle. Because of the sufficient confinement in the columns, the development of spalling was much more gradual than in the control specimens. With the progression of lateral excursions, concrete cover spalled off around the two peak points of the 4<sup>th</sup> lateral excursion cycle with a peak deflection of $2\Delta_y$. From the applied lateral load vs. displacement ($P_L-\delta_L$) curves, it could be observed that at the instant of cover spalling, the lateral deflection had an instantaneous increase of about 10-20%, combined with a slight decrease of lateral resistance.

After three lateral cycles, longitudinal cracks also occurred on the surface of columns along the
longitudinal bars in the most compressed zone, which indicated the tendency of buckling of the longitudinal bars. In these six specimens, the buckling lengths of longitudinal bars were greater than the corresponding spacing of spirals. Thus, the buckling bars, together with the expanding concrete core, pushed outwards onto the spirals. Finally, the spirals fractured in an explosive manner, resulting in the collapse of columns with the severe buckling of longitudinal bars and the crushing of the concrete core within plastic-hinge regions. Figures in Appendix A show these specimens after failure. The shifting-away of the most damaged section, i.e. the critical section, from the maximum moment section at the column-stub interface is due to the additional confining effect by the heavily reinforced concrete stub.

4.2.2 FRP-confined columns

The specimens in the second category consisted of seven columns which were initially deficient with respect to confining reinforcement and were retrofitted with transverse external FRP wrapping. These columns contained only the widely spaced steel spirals of #3@300 mm as transverse reinforcement within the potential plastic hinge regions before the FRP-retrofit.

The snapping sound of epoxy was heard clearly throughout testing, which mainly came from the gradual cracking of the epoxy between fibers in the FRP jackets within the plastic hinge regions. The initial occurrence of cracking and crushing of concrete were invisible inside the FRP jackets. All the FRP-confined columns behaved at first in a similar manner. There was no visible sign of damage in any of the columns until the 5th cycle of lateral excursion with the peak deflection of $2\Delta_y$. After that, localized color change in the FRP jackets was observed within the plastic hinge regions of columns adjacent to the column-stub interface, indicating the progressive deterioration of concrete. The discolored regions of the FRP jackets extended towards the tips of columns with the increasing lateral drift, reaching up to about twice the column diameter. Within the plastic-hinge regions of columns, the FRP jackets were dilated substantially with the incremental lateral deflection, which was verified by the transverse strain measured by the strain gauges mounted on the FRP surface.

Noticeable cracking was observed on the tension side of the columns right at the column-footing interface during the tests, and it was more pronounced for columns tested under low levels of axial load. This phenomenon was also observed during the tests of steel-confined columns, even though it was partially concealed by the spalling of concrete cover and not as clear as that
happened in the FRP-confined specimens. Recall that the longitudinal bars were extended to near the end of stubs in the specimens and sufficient anchorage length is provided for the longitudinal reinforcement. Despite that some yield penetration and slip of the tensile longitudinal bars cannot be avoided which results in additional column deflection.

In most cases, the rupture of fibers in the FRP jackets occurred near the column-stub interface during the last several cycles of lateral excursions. At first, the gradual rupture of some 3-5 mm wide FRP strips was observed, accompanied with a continuous snapping sound. Meanwhile, little loss of the shear capacity of columns could be detected in lateral shear vs. tip deflection curves. The specimens could still sustain the constant axial load and further lateral deflection. In the last cycle, the speeding-up rupture of the FRP strips, accompanied with the quick release of the stored tensile stress in the FRP composites, pronounced the failure of the columns. At this time, the opening of the ruptured FRP jackets had developed to a width of more than 100 mm. Thereafter, rapid concrete crushing resulted from the substantial loss of confinement. The gradual rupture process of FRP jackets was distinct from the observation in the concentric compression tests of FRP-confined 150×300 mm concrete cylinders, in which the collapse of concrete occurred with the rupture of FRP within a very short time in an explosive manner (Cui and Sheikh, 2010a).

The most extensive damage occurred within a segment of 100 mm to 420 mm from to the column-stub interface, which coincided with the location of fiber rupture. Similar to the steel-confined columns, the shifting-away of the critical section from the maximum moment section at the column-stub interface is attributed to the additional confining effect by the heavily reinforced concrete stub. Compared to the specimens under a low axial load level ($0.27P_o$), the columns subjected to higher axial load levels ($0.40P_o$ and $0.56P_o$) had larger damaged regions in the concrete core and larger area of ruptured FRP jackets.

The performance of compressive steel bars inside the FRP-retrofitted columns needs to be mentioned. Although only #3@300 mm spirals were provided initially, there was no sign of premature buckling of the longitudinal bars in this category of specimens. The cross sections of columns remained nearly unchanged throughout testing, until the failure owing to the rupture of the FRP jackets. Recall that during the tests of control specimens P40-NF-5 and P56-NF-10, it was the buckling of longitudinal reinforcement that led to severe spalling of concrete cover and
brittle column failure during the 4th cycle of lateral excursion. In the FRP-confined columns, even though the buckling of longitudinal bars was also observed after the collapse of columns, the FRP jackets showed the ability to prevent the premature buckling of the bars and ensure the ductile behaviour of columns.

As mentioned in Chapter 3, the overlap length at the end of FRP wrapping was 150 mm in the circumferential direction. No FRP debonding or rupture occurred in the overlap zone in any specimen. Additionally, at the locations where the FRP jackets were punctured by the threaded rods for mounting LVDTs, no premature FRP rupture was observed because of the strengthening in those positions by extra patches of FRP.

Overall, the inelastic behaviour of all the fifteen columns in this experimental research was dominated by flexure instead of shear because of the large shear span-depth ratio of 5.17. The specified axial load was applied on each column and kept constant during the testing until the specimens could not sustain the applied axial load and collapsed due to the excessive damage of core concrete, the buckling of longitudinal bars and the fracture or rupture of transverse reinforcement.

4.3 Results

4.3.1 Response of specimens

Each specimen represented a portion of a bridge or building column between the section of maximum moment and the point of contraflexure, as shown in Figure 4.1. To illustrate the behaviour of the specimens intuitively, the hysteresis loops of the lateral shear vs. tip deflection \((V-\Delta)\) relationship at the contraflexure point and the moment vs. curvature \((M-\phi)\) relationship at the critical section of each column are presented in Figure 4.3 to Figure 4.17. The experimental response of each specimen is also displayed graphically in Appendix B in the form of hysteresis loops of the applied lateral load vs. displacement \((P_L-\delta_L)\) at the loading point of the MTS actuator. The moment vs. tip deflection responses \((M-\Delta)\) are also presented in Appendix B because they are used by a few researchers in the analysis of columns.

Introduced first are the definitions of several key parameters shown in the schematic drawing of specimen in Figure 4.1. \textit{Tip deflection of column}, \(\Delta\), is the tangential deviation of the
contraflexure point of column (i.e. the distance from point $B$ to point $C$), which could be determined by the following expression:

$$\Delta = \delta_L \frac{a+b}{a}$$

(4-1)

where, $\delta_L = \text{deflection at the point of application of lateral load by the MTS actuator, measured by the vertical LVDTs along the column length during testing.}$ The following dimensions in the test setup were measured as:

- $a = 1022 \text{ mm}$
- $b = 2000 \text{ mm}$
- $c = 368 \text{ mm}$
- $H = 1473 \text{ mm}$

Figure 4.1 Schematic drawing of specimen
As mentioned in Chapter 3, even though the length of the concrete columns was 1473 mm, the test setup resulted in a shear span \( L = H + c = 1841 \) mm measured from the center of the right steel hinge at point \( B \) to the column-stub interface.

As illustrated in Figure 4.1, the lateral shear force \( V \) is conventionally defined as the shear force at the bottom of the column, which is perpendicular to the central axis \( AC \) of the column. The shear force \( V \) is approximately uniform along the column only if the lateral deflection (\( \Delta \)) is very small compared with the column length \( (L) \). However, in this experimental program the different concepts between the applied lateral force \( V' \) at the column tip and the lateral shear force \( V \) at the bottom of the column should be discerned. Since the axial load \( P \) is applied constantly along the original axis \( AB \) instead of the moved axis \( AC \), it is not perpendicular to the stub-column interface all the time during the laterally cyclic loading process. The transverse component of force \( P \) should be subtracted from the applied lateral force \( V' \) when determining the resultant shear force \( V \).

The applied lateral force \( V' \) is estimated from the applied lateral force \( P_L \) by the MTS actuator as

\[
V' = P_L \frac{a}{a + b}
\]  

(4-2)

The inclination \( \theta \) of the column-stub interface from its original position is

\[
\theta = \frac{\Delta}{a + b}
\]  

(4-3)

Thus, the shear force \( V \) at the bottom of column is determined as

\[
V = V' \cos \theta - P \sin \theta
\]  

(4-4)

When the lateral deflection \( \Delta \) and applied axial load \( P \) are large, the difference between \( V \) and \( V' \) can be significant. In order to demonstrate this issue, the test results of column P56-4GF-15 are presented in Figure 4.2 as an example, which sustained an axial load \( P = 0.56P_o \) (2661 kN). At the first peak offset point in the last lateral excursion cycle, the lateral load by the MTS actuator was measured as \( P_L = 295 \) kN when the lateral deflection \( \Delta \) reached 173 mm. At this instant, the shear force \( V \) at the bottom of the column was estimated as equal to -53 kN according to Eq.(4-4), while the applied lateral force \( V' \) at the column tip was about 100 kN according to Eq.(4-2), which can also be observed in Figure 4.2 at the first peak offset point in the last lateral
cycle. The difference between the two forces $V$ and $V'$ was very impressive. On the other hand, it is also noticed that the resultant compression force along the column axis $AC$ was 2662 kN at this moment, which was similar to the applied axial load $P = 2661$ kN along the original axis $AB$. Thus, the axial force along the column could approximately be deemed to be constant and uniform during the process of laterally cyclic loading.

Sheikh and Khoury (1993) indicated that the sectional behaviour represented by the moment vs. curvature relationship was the primary concern of seismic behaviour of the column, because the lateral tip deflection $\Delta$ can be estimated by the integration of curvature along the column length. Especially, most of the inelastic lateral deflection comes from the plastic deformation concentrating within the plastic hinge region of the column during severe earthquakes. Thus, the moment vs. curvature relationship at the critical section was presented for each column in this experimental program. As noted earlier, the critical section may be shifted away from the section with the highest moment right adjacent to the stub, as indicated by the most damaged region.

**Moment at the critical cross section**, $M$, is the sum of the primary moment caused by the laterally cyclic shear $V$ and the secondary moment by the axial load $P$, and can be estimated by either of the following two expressions:

$$M = V(L - D_{md}) + P\Delta$$  \hspace{1cm} (4-5)

or

$$M = V'(L - D_{md}) + P\delta_{md}$$  \hspace{1cm} (4-6)
where,

\[ P \quad = \quad \text{applied axial load on column;} \]
\[ L \quad = \quad \text{shear span of column;} \]
\[ D_{md} \quad = \quad \text{distance from column-stub interface to the critical section;} \]
\[ \delta_{md} \quad = \quad \text{lateral deflection at the critical section, estimated from the measured deflection shape of column.} \]

The measured value of \( D_{md} \) of each specimen is reported later in Table 4.2. The curvature \( \phi \) at the critical section was evaluated from the deformation of the core concrete of the column, measured by the LVDTs mounted in the failure zone. However, the difficulty should be recognised in obtaining the curvature very accurately during testing due to the arbitrary gauge length of LVDTs. In this experimental program, the deformation of the column core was measured by the LVDTs with gauge length of about 100 mm. The measured curvature at large deformations should be considered as the approximate average over the gauge length. Additionally, the threaded rods on which the LVDTs were mounted could not be held firmly by the core concrete which was severely damaged when the column was close to failure.
Figure 4.3 Experimental hysteresis response of column P27-NF-1

(a) Lateral shear vs. tip deflection ($V$-$\Delta$) relationship

(b) Moment vs. curvature ($M$-$\phi$) relationship at critical section
Figure 4.4 Experimental hysteresis response of column P27-NF-2

(a) Lateral shear vs. tip deflection ($V$-$\Delta$) relationship

(b) Moment vs. curvature ($M$-$\phi$) relationship at critical section
(a) Lateral shear vs. tip deflection ($V$-$\Delta$) relationship

(b) Moment vs. curvature ($M$-$\phi$) relationship at critical section

Figure 4.5 Experimental hysteresis response of column P27-1CF-3
(a) Lateral shear vs. tip deflection ($V$-$\Delta$) relationship

(b) Moment vs. curvature ($M$-$\phi$) relationship at critical section

Figure 4.6 Experimental hysteresis response of column P27-2GF-4
(a) Lateral shear vs. tip deflection ($V$-$\Delta$) relationship

(b) Moment vs. curvature ($M$-$\phi$) relationship at critical section

Figure 4.7 Experimental hysteresis response of column P40-NF-5
Figure 4.8 Experimental hysteresis response of column P40-NF-6

(a) Lateral shear vs. tip deflection ($V$-$\Delta$) relationship

(b) Moment vs. curvature ($M$-$\phi$) relationship at critical section
Fig. 4.9 Experimental hysteresis response of column P40-NF-7

(a) Lateral shear vs. tip deflection ($V$-$\Delta$) relationship

(b) Moment vs. curvature ($M$-$\phi$) relationship at critical section
Figure 4.10 Experimental hysteresis response of column P40-1CF-8

(a) Lateral shear vs. tip deflection ($V$-$\Delta$) relationship

(b) Moment vs. curvature ($M$-$\phi$) relationship at critical section
(a) Lateral shear vs. tip deflection ($V-\Delta$) relationship

(b) Moment vs. curvature ($M-\phi$) relationship at critical section

Figure 4.11 Experimental hysteresis response of column P40-1GF-9
Figure 4.12 Experimental hysteresis response of column P56-NF-10
(a) Lateral shear vs. tip deflection ($V$-$\Delta$) relationship

(b) Moment vs. curvature ($M$-$\phi$) relationship at critical section

Figure 4.13 Experimental hysteresis response of column P56-NF-11
Figure 4.14 Experimental hysteresis response of column P56-NF-12

(a) Lateral shear vs. tip deflection ($V$-$\Delta$) relationship

(b) Moment vs. curvature ($M$-$\phi$) relationship at critical section
Figure 4.15 Experimental hysteresis response of column P56-2CF-13

(a) Lateral shear vs. tip deflection ($V-\Delta$) relationship

(b) Moment vs. curvature ($M-\phi$) relationship at critical section
(a) Lateral shear vs. tip deflection ($V$-$\Delta$) relationship

(b) Moment vs. curvature ($M$-$\phi$) relationship at critical section

Figure 4.16 Experimental hysteresis response of column P56-3GF-14
(a) Lateral shear vs. tip deflection ($V$-$\Delta$) relationship

(b) Moment vs. curvature ($M$-$\phi$) relationship at critical section

Figure 4.17 Experimental hysteresis response of column P56-4GF-15
4.3.2 Ductility parameters

It is easy to define ductility parameters for an linearly elastic-perfectly plastic behaviour. However, the nonlinear behaviour of reinforced concrete elements under seismic load is generally not linearly elastic-perfectly plastic. Consequently, there is no universal definition of ductility for reinforced concrete columns in the available scientific literature.

In the research of the seismic behaviour of structures, three parameters are mainly used to describe the ductility levels of concrete columns, which include the curvature ductility factor $\mu_\phi$, the displacement ductility factor $\mu_\Delta$ and the lateral drift ratio $\delta$. In order to define these parameters in a simple, reasonable and unified manner, a set of definitions are used in this study which are introduced as follows.

1. **Nominal flexural strength, $M_n$**

Nominal flexural strength, $M_n$, is defined as the theoretical flexural strength of a reinforced concrete column with unconfined concrete and subjected to a certain level of axial load. The amount of $M_n$ can be estimated through sectional analysis of the cross section of column, in which the linearly elastic-perfectly plastic stress-strain relationship is assumed for the longitudinal steel under both tension and compression, while the tensile strength of concrete, the tension stiffening of reinforced concrete (i.e. the contribution of the concrete in tension between cracks to the stiffness of element), the strain-hardening and compressive buckling of longitudinal steel are ignored. The stress-strain relationship proposed by Attard and Setunge (1996) is used for the unconfined concrete, with the ultimate strain of 0.0035 (CAN/CSA A23.3-04). All resistance reduction factors are taken as unity. In fact, similar estimation of $M_n$ can be obtained by using other constitutive relationships proposed for unconfined concrete.

2. **Yield curvature, $\phi_y$**

Yield curvature, $\phi_y$, is defined as the curvature corresponding to the nominal flexural strength $M_n$ along a straight line joining the origin and a point of 65%$M_n$ on the ascending branch of the theoretical moment vs. curvature curve. To get the theoretical moment vs. curvature curve, a sectional analysis shall be conducted with a series of incremental curvatures until failure, in which the material properties are the same as in the definition of $M_n$. According to this definition,
the amount of yield curvature has no relation with the level of transverse confinement.

There are many different definitions of yield curvature and yield deflection used by different researchers. The definition used here is adopted from previous research and it allows comparison with earlier tests. The ACI 374 document on Protocol of Testing also supports this assertion. As observed in seismic tests of columns, the deflection of columns is almost linear up to $0.65M_n$ and $0.65V_n$ in most cases. Furthermore, these two points are approximately the upper limit in columns design.

3. \textit{Ultimate curvature, }$\phi_u$

Ultimate curvature, $\phi_u$, is defined as the curvature when the post-peak flexural capacity drops to 80% of the nominal flexural strength $M_n$ or the occurrence of the column failure under axial load, whichever occurs first. Because of the strength redundancy in a structural system, it is normally deemed that the column still has sufficient reserve of the load carrying capacity if its strength decay is no more than 20% of the nominal strength $M_n$.

4. \textit{Curvature ductility factor, }$\mu_\phi$

Curvature ductility factor, $\mu_\phi$, is defined as the ratio of the ultimate curvature $\phi_u$ to the yield curvature $\phi_y$ of column cross section.

5. \textit{Nominal lateral force capacity, }$V_n$

Nominal lateral force capacity, $V_n$, of the cantilever column is determined by

$$V_n = \frac{(M_n - P \cdot \Delta_y)}{L} \quad (4-7)$$

where:

\begin{align*}
P & = \text{constant axial load supported by column;} \\
L & = \text{shear span of column;} \\
\Delta_y & = \text{yield lateral deflection.}
\end{align*}

In this definition, the $P$-$\Delta$ effect has been taken into account here by subtracting the term $P \cdot \Delta_y$ from $M_n$. 
6. **Yield deflection, Δ_y**

Yield deflection, Δ_y, is defined as the lateral deflection corresponding to the nominal lateral force capacity \( V_n \) along a straight line joining the origin and a point of 65%\( V_n \) on the ascending branch of the experimental lateral shear vs. tip deflection envelope curve, averaged in two directions. In the theoretical analysis, yield deflection can be determined in the same way based on the theoretical lateral shear vs. tip deflection curve.

7. **Ultimate deflection, Δ_u**

Ultimate deflection, Δ_u, is defined as the lateral deflection when the post-peak lateral force capacity of the column drops to 80% of the nominal lateral force capacity \( V_n \) or when the column cannot sustain the axial load anymore, whichever occurs first.

8. **Displacement ductility factor, \( \mu_\Delta \)**

Displacement ductility factor, \( \mu_\Delta \), is defined as the ratio of the ultimate deflection Δ_u to the yield deflection Δ_y at the tip of the cantilever column.

9. **Lateral drift ratio, \( \delta \)**

Lateral drift ratio, \( \delta \), is defined as the ratio of the ultimate lateral deflection Δ_u to the shear span of column \( L \). This parameter is the rotation of column chord corresponding to Δ_u and can be used to represent the deformability of column. But it has no relation with the yield deflection Δ_y.

It should be noted that the above introduced parameter definitions are not the same as the sets of definitions for ductility parameters used by Sheikh and Khoury (1993) or Bae (2005), in which the actually measured strength was used to evaluate the yield curvature, ultimate curvature, yield deflection and ultimate deflection. In the current research, the concept of nominal strength is used to determine the ductility parameters.

Since the nominal strength of the column is based on the properties of the unconfined concrete, it would most likely be less than the actually achievable strength of the confined column. For steel-confined columns, the difference between the measured strength and the nominal strength may not be so significant. However, for FRP-confined columns, the actual measured strength may be
much higher than the nominal strength, which will be demonstrated later in the experimental and theoretical analyses. Therefore, the ductility factors evaluated based on the current definitions are higher than those based on the definitions used by Sheikh and Khoury (1993) or Bae (2005). The differences are not significant for steel-confined columns but they are substantial for the FRP-confined columns.

The experimental results of curvature and displacement ductility factors of all the specimens in this experimental program are presented in Table 4.1, while the lateral drift ratios are presented in Table 4.8. The corresponding results of the specimens tested by Sheikh and Yau (2002) are also included in these tables for the further analysis.

### 4.3.3 Critical section

During testing, the moment varied along the length of the cantilever column. Therefore, the location of the critical section, i.e. the most damaged section of the column, is very important for studying the sectional seismic behaviour. Even though the maximum moment in the column was at the column-stub interface, the failure occurred away from this interface in all the specimens. In Table 4.2 the locations of the critical sections in all fifteen specimens are presented. Also listed in this table are the measured maximum shear force $V_{\text{max}}$ and the corresponding maximum moment $M_{\text{max}}$ at the critical section. The figures in Appendix A present pictures of all the columns after failure, which clearly show the damaged zones.
<table>
<thead>
<tr>
<th>Researcher Specimen</th>
<th>Axial load $P/P_o$</th>
<th>Index $I_e = f_i/f_c$</th>
<th>Curvature ductility factor $\phi_y$ (rad/km)</th>
<th>Curvature ductility factor $\phi_u$ (rad/km)</th>
<th>Displacement ductility factor $\mu_\phi$</th>
<th>$\Delta_y$ (mm)</th>
<th>$\Delta_u$ (mm)</th>
<th>$\mu_\Delta$</th>
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<td>Current tests</td>
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<tr>
<td>P27-NF-1</td>
<td>0.27</td>
<td>0.031</td>
<td>12.4</td>
<td>156.8</td>
<td>12.6</td>
<td>17.0</td>
<td>58.5</td>
<td>3.4</td>
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<tr>
<td>P27-NF-2</td>
<td>0.27</td>
<td>0.051</td>
<td>12.4</td>
<td>245.8</td>
<td>19.8</td>
<td>17.0</td>
<td>65.1</td>
<td>3.8</td>
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<tr>
<td>P27-1CF-3</td>
<td>0.27</td>
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<td>12.4</td>
<td>234.6</td>
<td>18.9</td>
<td>17.0</td>
<td>87.9</td>
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<td>P27-2GF-4</td>
<td>0.27</td>
<td>0.131</td>
<td>12.4</td>
<td>217.6</td>
<td>17.5</td>
<td>17.0</td>
<td>83.0</td>
<td>4.9</td>
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<td>P40-NF-5</td>
<td>0.40</td>
<td>--</td>
<td>9.8</td>
<td>39.8</td>
<td>4.1</td>
<td>14.1</td>
<td>30.6</td>
<td>2.2</td>
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<tr>
<td>P40-NF-6</td>
<td>0.40</td>
<td>0.051</td>
<td>9.8</td>
<td>168.7</td>
<td>17.2</td>
<td>12.6</td>
<td>40.1</td>
<td>3.2</td>
</tr>
<tr>
<td>P40-NF-7</td>
<td>0.40</td>
<td>0.072</td>
<td>9.8</td>
<td>159.5</td>
<td>16.3</td>
<td>14.4</td>
<td>57.3</td>
<td>4.0</td>
</tr>
<tr>
<td>P40-1CF-8</td>
<td>0.40</td>
<td>0.120</td>
<td>9.8</td>
<td>168.2</td>
<td>17.2</td>
<td>11.3</td>
<td>78.5</td>
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<td>P40-1GF-9</td>
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<td>8.6</td>
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<td>10.8</td>
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<td>0.236</td>
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<td>0.243</td>
<td>8.6</td>
<td>229.0</td>
<td>26.6</td>
<td>12.0</td>
<td>86.1</td>
<td>7.2</td>
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<td>P56-4GF-15</td>
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<td>0.275</td>
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<td>249.3</td>
<td>29.0</td>
<td>10.6</td>
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<td>8.8</td>
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<td>Sheikh and Yau (2002)</td>
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<td></td>
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<td>0.068</td>
<td>8.8</td>
<td>35.1</td>
<td>4.0</td>
<td>19.8</td>
<td>39.7</td>
<td>2.0</td>
</tr>
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<td>S-2NT</td>
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<td>0.068</td>
<td>12.9</td>
<td>162.0</td>
<td>12.6</td>
<td>27.7</td>
<td>74.9</td>
<td>2.7</td>
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<tr>
<td>S-3NT</td>
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<td>--</td>
<td>8.8</td>
<td>18.4</td>
<td>2.1</td>
<td>15.1</td>
<td>36.2</td>
<td>2.4</td>
</tr>
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<td>S-4NT</td>
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<td>--</td>
<td>12.9</td>
<td>44.0</td>
<td>3.4</td>
<td>26.7</td>
<td>55.2</td>
<td>2.1</td>
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<td>ST-2NT</td>
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<td>0.107</td>
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<td>94.6</td>
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<td>14.3</td>
<td>79.0</td>
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<td>0.053</td>
<td>13.0</td>
<td>155.0</td>
<td>11.9</td>
<td>24.7</td>
<td>100.9</td>
<td>4.1</td>
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Table 4.1 Measured ductility factors of specimens
Table 4.2 Measured strength and damaging sketch of specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Transverse confinement</th>
<th>(L_{dr}) (mm)</th>
<th>(D_{dr}) (mm)</th>
<th>(D_{md}) (mm)</th>
<th>(V_{max}) (kN)</th>
<th>(M_{max}) (kNm)</th>
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</thead>
<tbody>
<tr>
<td>P27-NF-1</td>
<td>#3@150</td>
<td>370</td>
<td>70</td>
<td>265</td>
<td>100</td>
<td>204</td>
</tr>
<tr>
<td>P27-NF-2</td>
<td>#3@100</td>
<td>310</td>
<td>20</td>
<td>160</td>
<td>101</td>
<td>220</td>
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<tr>
<td>P27-1CF-3</td>
<td>1-layer CFRP</td>
<td>180</td>
<td>0</td>
<td>110</td>
<td>119</td>
<td>264</td>
</tr>
<tr>
<td>P27-2GF-4</td>
<td>2-layer GFRP</td>
<td>190</td>
<td>0</td>
<td>80</td>
<td>105</td>
<td>251</td>
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<tr>
<td>P40-NF-5</td>
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<td>430</td>
<td>80</td>
<td>260</td>
<td>93</td>
<td>180</td>
</tr>
<tr>
<td>P40-NF-6</td>
<td>#3@100</td>
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<td>275</td>
<td>100</td>
<td>240</td>
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<tr>
<td>P40-NF-7</td>
<td>#3@75</td>
<td>332</td>
<td>100</td>
<td>250</td>
<td>108</td>
<td>229</td>
</tr>
<tr>
<td>P40-1CF-8</td>
<td>1-layer CFRP</td>
<td>260</td>
<td>30</td>
<td>150</td>
<td>99</td>
<td>262</td>
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<td>P40-1GF-9</td>
<td>1-layer GFRP</td>
<td>308</td>
<td>60</td>
<td>203</td>
<td>116</td>
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<td>P56-NF-10</td>
<td>#3@300</td>
<td>370</td>
<td>150</td>
<td>300</td>
<td>91</td>
<td>193</td>
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<td>P56-NF-11</td>
<td>10M@100</td>
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<td>100</td>
<td>260</td>
<td>95</td>
<td>203</td>
</tr>
<tr>
<td>P56-NF-12</td>
<td>10M@175</td>
<td>350</td>
<td>70</td>
<td>270</td>
<td>93</td>
<td>197</td>
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<tr>
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<td>2-layer CFRP</td>
<td>265</td>
<td>20</td>
<td>160</td>
<td>98</td>
<td>331</td>
</tr>
<tr>
<td>P56-3GF-14</td>
<td>3-layer GFRP</td>
<td>315</td>
<td>60</td>
<td>216</td>
<td>110</td>
<td>337</td>
</tr>
<tr>
<td>P56-4GF-15</td>
<td>4-layer GFRP</td>
<td>305</td>
<td>114</td>
<td>215</td>
<td>119</td>
<td>364</td>
</tr>
</tbody>
</table>

Definitions:
\(L_{dr}\) = Length of most damaged region in tested columns;
\(D_{dr}\) = Distance from stub face to right end of the most damaged region;
\(D_{md}\) = Distance from stub face to the critical section;
\(V_{max}\) = Maximum shear force of column at failure;
\(M_{max}\) = Maximum moment at the critical section;
4.4 Discussion of experimental results

The experimental results show that the seismic behaviour of confined concrete columns is significantly affected by the variables such as the spacing of spiral reinforcement, the type and amount of transverse FRP, and the axial load level. Effects of these variables will be discussed in the following sections. Furthermore, some experimental observations will also be discussed regarding the additional confinement by the stub, the buckling behaviour of longitudinal steel, the increase of the ultimate compressive strain of concrete, the maximum strain in FRP wrapping, equivalent plastic hinge length, and the strength enhancement of columns due to confinement.

In the following sections, the level of transverse confinement to each column is represented by the confinement index $I_e = f_l / f_c'$, which is the ratio of the effective confining pressure $f_l$ in the column to the compressive strength of unconfined concrete $f_c'$, as shown in Table 4.1. The determination of the effective confining pressure $f_l$ will be introduced in the following sections for columns with transverse steel- or FRP-confinement.

4.4.1 Effect of spiral reinforcement

In steel-confined columns, the effective confining pressure $f_l$ applied to the core concrete mainly depends on the amount and strength of the transverse reinforcement, and steel configuration (Sheikh and Uzumeri, 1982; Mander et al. 1988). The relationship between the confinement level and behaviour of the column can be studied by comparing the steel-confined specimens in which only the transverse reinforcement was varied (Figure 4.3 to 4.17). Figure 4.18 compares the curvature and displacement ductility factors for specimens in three groups. The first group of specimens was tested under axial loads of $0.25P_o - 0.27P_o$, while the second and third groups were subjected to axial loads of $0.40P_o$ and $0.50P_o - 0.56P_o$, respectively. The specimens within each group were similar in all respects except the level of transverse confinement.

In the first group, specimens S-4NT, P27-NF-1, P27-NF-2 and S-2NT have #3 spirals at spacing of 300 mm, 150 mm, 100 mm and 80 mm, respectively. Control specimen S-4NT behaved in a very brittle manner with a curvature ductility factor $\mu_\phi$ of 3.4 and a displacement ductility factor $\mu_\Delta$ of 2.1. The ductility factors of the other three specimens were increased significantly due to the increase of transverse steel contents accompanied with the reduction of spiral spacing.
In the second group, the #3 spirals in specimens P40-NF-5, P40-NF-6 and P40-NF-7 were spaced at 300 mm, 100 mm, and 75 mm respectively. Up to failure, the columns achieved the displacement ductility factors $\mu_\Delta$ of 2.2, 3.2 and 4.0; and their curvature ductility factors $\mu_\phi$ were 4.0, 17.2 and 16.3, respectively. The improvement of ductility factor is the direct result of the increased transverse confinement level.

Similar observation can also be found in the third group of steel-confined columns by the comparison of testing results. Specimens P56-NF-10, P56-NF-11 and P56-NF-12 modeled the design of the non-ductile, moderately-ductile and ductile columns according to CAN/CSA-A23.3-04, respectively. Control specimen P56-NF-10 just obtained a factor $\mu_\Delta$ of 2.1 and a factor $\mu_\phi$ of 1.8. Specimen P56-NF-11, transversely confined by 10M@100 mm, had a $\mu_\Delta$ factor of 3.3 and a $\mu_\phi$ factor of 12.8. Specimen P56-NF-12, with 10M@75 mm spirals, displayed a $\mu_\Delta$ of 3.7 and a $\mu_\phi$ of 18.2.

The spacing of spirals is the key to prevent the premature buckling of the longitudinal rebars in steel-confined columns. The longitudinal bars of all specimens in this experimental program had a diameter of $d_b=25$ mm. In P40-NF-5 and P56-NF-10, spirals were spaced at 300 mm, i.e. 12 times of $d_b$. These two columns failed in a very brittle manner owing primarily to the premature buckling of longitudinal bars and the crushing of core concrete, with the measured $\mu_\phi$ of no more.
than 4.1. On the contrary, in columns with the spiral spacing varying from 150 mm to 75 mm, no premature bar buckling occurred and the columns failed in a ductile manner with \( \mu_\phi \) larger than 12.6. This phenomenon has also been observed in square and rectangular columns by Bayrak and Sheikh (2001), who demonstrated that premature buckling could be prevented under cyclic loading only if the \( s/d_b \) ratio was no more than 6. If the bar buckling is ignored and the longitudinal steel bars are assumed to have the same stress-strain relationship when subjected to tension and compression, it may lead to unconservative predictions of the seismic performance of the columns with the ratio \( s/d_b \) more than 6.

4.4.2 Effect of transverse FRP jackets

The compressive strength and deformability of concrete can be significantly improved by the transverse FRP confinement, which is the major reason for the enhancement of strength, ductility and energy dissipation capacity of FRP-retrofitted concrete columns.

The effective confining stress, \( f_c \), of each FRP-confined specimen in this experimental program is estimated according to the Cui and Sheikh model (Cui and Sheikh, 2010b). The strain efficiency coefficient \( k_e \) in the FRP jackets is taken as actual measured values for the specimens in this experimental program, while the proposed value \( k_e = 0.6 \) by Cui and Sheikh model is used for the specimens reported by Sheikh and Yau (2002) owing to a lack of corresponding information. The effectiveness of the FRP jackets is evaluated by comparing the test results of the three groups of specimens, which were subjected to axial load of \( 0.23P_o - 0.27P_o, 0.40P_o \) and \( 0.50P_o - 0.56P_o \), respectively. The relationships between the confinement index \( I_e = f_i/f_c \) and the ductility factors are presented in Figure 4.19 for the control specimens and the FRP-confined specimens listed in Table 4.1. The hysteresis loops of shear vs. tip deflection (\( V-\Delta \)) and moment vs. curvature (\( M-\phi \)) relationships in Section 4.3 and the applied lateral force vs. deflection (\( P_L-\delta_L \)) in Appendices B display the responses of these columns.

The first group consisted of columns S-4NT, P27-1CF-3 and P27-2GF-4, subjected to axial loads of \( 0.23P_o - 0.27P_o \). The control specimen S-4NT was similar to P27-1CF-3 and P27-2GF-4 in all respects except that it had no FRP-confinement. As shown in Table 4.1, Specimen S-4NT obtained a curvature ductility factor \( \mu_\phi \) of only 3.4 and a displacement ductility factor \( \mu_\Delta \) of 2.1. Specimen P27-1CF-3 was retrofitted with one layer of CFRP jacket and had the \( \mu_\phi \) of 18.9 and
the $\mu_\Delta$ of 5.2. Similarly, two layers of GFRP jackets in Specimen P27-2GF-4 improved its ductility factors $\mu_\phi$ and $\mu_\Delta$ to 17.5 and 4.9, respectively. The curvature ductility factors were increased by the FRP retrofit to about five times. Additionally, the enhancement of ductility in specimens P27-1CF-3 and P27-2GF-4 were quite analogous, which indicated that two layers of GFRP can have a similar confinement effect as one layer of CFRP jacket. The FRP coupon testing has shown that one layer of CFRP jacket had about twice the ultimate tensile strength ($f_{Fu}$) and three times the tensile stiffness ($E_F$) of one layer of GFRP jacket. It appeared that the effectiveness of FRP wrapping to enhance the column ductility related more closely to its ultimate tensile strength than its stiffness.

The second set of specimens include columns P40-NF-5, P40-1CF-8 and P40-1GF-9, and were all subjected to a constant axial load of $0.40P_o$. Here specimen P40-NF-5 acted as a control specimen, while P40-1CF-8 and P40-1GF-9 were retrofitted by one layer of CFRP and GFRP wrapping, respectively. The measured $\mu_\phi$ factors of P40-NF-5, P40-1CF-8 and P40-1GF-9 were 4.1, 17.2 and 10.7, respectively, while their $\mu_\Delta$ factors were 2.2, 6.9 and 6.3, respectively. The improvement of ductility attributed to the FRP retrofit was obvious.

![Curvature ductility and Displacement ductility](image)

(a) Curvature ductility (b) Displacement ductility

Figure 4.19 Relationships between ductility factors and confinement in FRP-confined specimens

The third group of specimens consisted of P56-NF-10, P56-2CF-13, P56-3GF-14, P56-4GF-15, S-3NT, ST-2NT and ST-3NT, which were subjected to the axial loads of $0.50P_o$ to $0.56P_o$. The
control specimens P56-NF-10 and S-3NT were transversely reinforced solely by #3@300 mm spirals and behaved in a very brittle manner. The FRP-retrofitted specimens P56-2CF-13, P56-3GF-14, P56-4GF-15, ST-2NT and ST-3NT could all sustain the incremental lateral excursion in a ductile manner, while the measured $\mu_\Delta$ ranged from 3.9 to 8.8 and the $\mu_\phi$ between 8.9 and 29.0. Moreover, the FRP-retrofitted columns developed very stable hysteresis loops of moment vs. curvature curves. Their moment carrying capacity kept increasing and showed no degradation until the final failure caused by the rupture of FRP.

No sign of premature buckling of the longitudinal steel bars was observed in the tests until the rupture of FRP jackets and the subsequent concrete crushing, even though only #3@300 mm steel spirals were provided in all the FRP-confined columns. This indicated that the FRP wrapping was able to provide sufficient transverse restraint against premature bar buckling. Meanwhile, it should also be realized that the longitudinal bars with buckling trend applied an outward push onto the FRP wrapping, resulting in some additional tensile stress in the FRP jackets. The interaction between the longitudinal bars and the FRP wrapping may lead to somewhat early rupture of the FRP jackets.

4.4.3 Effect of axial load level

The detrimental effect of high axial load on the curvature ductility factor has been well documented in the past two decades for both steel-confined and FRP-confined concrete columns (Sheikh and Khoury, 1993, Iacobucci et al, 2003, Memon and Sheikh, 2005). The relationship among the applied axial load, amount of transverse confinement and ductility performance of columns can be evaluated through the comparison of specimens which were similar in all respects except for the axial load levels. Three similar specimens, P27-1CF-3, P40-1CF-8 and ST-3NT were tested under axial load levels of $0.27P_o$, $0.4P_o$ and $0.50P_o$, respectively. The measured values of their curvature ductility factors $\mu_\phi$ were 18.9, 17.2, and 8.9, respectively, which were approximately inversely-proportional to the axial load levels. The displacement ductility factors $\mu_\Delta$ of the three columns are 5.2, 6.9 and 3.9, respectively. The higher $\mu_\Delta$ in P40-1CF-8 compared with P27-1CF-3 is somewhat unusual but seems to result from higher confinement index for that column.
On the other hand, for columns to achieve similar curvature ductility factor, the required confinement was significantly increased with the applied axial load level. Specimens P27-1CF-3 and P56-2CF-13 had similar ductility performance with column P56-2CF-13 showing a slightly better performance. Specimen P27-1CF-3 was confined with one layer of CFRP wrapping and subjected to axial load of $0.27P_o$, while P56-2CF-13 under axial load of $0.56P_o$ was retrofitted by two layers of CFRP wrapping. The effect of higher axial load was compensated by the increased confinement. Similar observation can also be made by the comparison between the other column sets. Thus, the design procedures for the columns to determine the required transverse confinement should incorporate the effect of the axial load level if the curvature ductility factor is taken as the design criterion.

It was found that the effectiveness of confinement to enhance the ductility of columns varied depending on the ductility parameter used. From the standpoint of curvature ductility and the maximum number of lateral displacement cycles until final column failure, increasing confinement reinforcement almost always improved the ductility of columns. For example, columns P56-2CF-13 and P56-4GF-15 were tested under similar loading conditions and identical in all respects except that the former was retrofitted with 2 layers of CFRP wraps while the latter with 4 layers of GFRP wraps. The measured curvature ductility factor improved from 21.0 of P56-2CF-13 to 29.0 of P56-4GF-15 as a result of the increased confinement. Until failure, the column P56-4GF-15 had already endured 17 lateral cycles instead of the 10 cycles of P56-2CF-13. However, the displacement ductility and lateral drift ratio of these two columns were almost equal. From a close study of their $V$-$\Delta$ responses in Figures 4.15 and 4.17, it could be seen that the slope of descending branch of the $V$-$\Delta$ envelope curves was mainly controlled by the $P$-$\Delta$ effect. The shapes of two $V$-$\Delta$ curves were similar before the post-peak branches dropped to $0.8V_n$. After 9 loops of lateral cyclic loading, the shear capacities of both columns had already dropped below $0.8V_n$, therefore the further incremental lateral deflection has no contribution to the displacement ductility factor and drift ratio. It is thus obvious that above a certain threshold, additional confinement can efficiently enhance only the curvature ductility and energy dissipation capacity of the plastic hinge region of columns, but not their displacement ductility and lateral drift capacity due to the severe $P$-$\Delta$ effect, especially for columns with large shear span-depth ratio and high axial load level.
4.4.4 Effect of stub

The critical sections of all the columns were located at a short distance away from the column-stub interface, as shown in Figures in Appendix A and Table 4.2. In the previous experiments conducted by Sheikh and Khoury (1993), Sheikh et al. (1994), and Sheikh and Yau (2002), similar observations were also reported. The strong stub provided additional confinement to the adjacent column segment, which delayed the transverse expansion and crack spread in concrete. Thus, the flexural strength of the column segment adjacent to the column-stub interface increased and the failure was shifted away to a nearby weaker segment.

Based on the experimental observation on steel-confined columns, Sheikh and Khoury (1993) suggested that the confinement effect of the stubs was significant within the region of a length of about the column section dimension. Three observations are made in this experimental program with respect to the damaged zone. Firstly, for all the fifteen specimens listed in Table 4.2, none of the distances, \( D_{md} \), between the critical section and column-stub interface was as large as the column’s diameter. The measured \( D_{md} \) ranged from 80 mm to 300 mm, with an average of 211 mm, which was about half of the column’s diameter. Secondly, it was observed that the average value of \( D_{md} \) in steel-confined columns was greater than that in the FRP-confined columns. The average \( D_{md} \) was 255 mm for the eight steel-confined columns and 160 mm for the seven FRP-confined columns. Lastly, in the steel-confined columns, \( D_{md} \) did not change much with axial load. However, for the FRP-confined columns, the measured \( D_{md} \) changed with the axial load level. In the first group, the two FRP-confined specimens under axial load of \( 0.27P_o \) showed an average \( D_{md} \) of 95 mm. When the axial load was increased to \( 0.40P_o \) in the second group of specimens, the two FRP-confined columns had an average \( D_{md} \) of 177 mm. Furthermore, for the third group under the highest axial load level of \( 0.56P_o \), the average \( D_{md} \) was 193 mm. These results clearly showed that, for the FRP-confined columns, the influence of the additional confinement by the stub increased with the axial load and pushed the critical section farther away from the column-stub interface.

While taking advantage of the increased flexural strength of the column section due to the extra confinement by the stub, its downside should also be recognized. For the capacity design method which is widely applied in current seismic design, this phenomenon leads to an increase of the demand of shear resistance of columns due to the reduced shear span. Even though it has been
postulated that the moment capacity of the plastic hinge is not changed by the stub effect (Sheikh and Khoury, 1993), the distance between the plastic hinge and the contraflexure point in column is shortened. The demand of shear resistance should be estimated based on the shortened shear span to avoid deficient shear design of columns.

### 4.4.5 Efficiency of transverse FRP jackets

The maximum confinement in FRP-confined columns depends on the achievable ultimate strain of FRP wrapping or so-called circumferential failure strain $\varepsilon_{F,1}$. In seismic tests of FRP-confined columns, the measured $\varepsilon_{F,1}$ has been reported to be lower than the FRP rupture strain $\varepsilon_{F,2}$ measured in the FRP coupon tests carried out in accordance with ASTM D3039/D3039M or other similar standards. The ratio of the actual failure strain $\varepsilon_{F,1}$ to the FRP coupon rupture strain $\varepsilon_{F,2}$ is referred to as the efficiency coefficient of FRP strain $k_{\varepsilon} = \varepsilon_{F,1} / \varepsilon_{F,2}$, which was suggested to be 0.637 for CFRP and 0.744 for GFRP wrapping by Lam and Teng (2004) based on concentric compression tests. From the measured maximum tensile strain in FRP jackets presented in Table 4.3, it was found that the coefficient $k_{\varepsilon}$ was 0.876 for CFRP and 0.796 for GFRP jackets in the seven FRP-confined specimens in this experimental research. The lowest $k_{\varepsilon}$ was measured as 0.719 in the test of P27-2GF-4 and the highest was 0.907 for specimen P40-1CF-8.

It is not quite clear why the measured rupture strain of FRP in columns is lower than that in coupons. As discussed by Lam and Teng (2004), the efficiency of FRP jackets may be affected by the column curvature, the number of layers of FRP, the presence of splices and the concentration of strain due to the localized fracture in concrete. Based on concentric compression tests of large and small size FRP-confined columns, Sheikh et al. (2007) have suggested that a major reason is that the reported strains $\varepsilon_{F,1}$ are not always measured at the location of rupture. While the rupture may be taking place at a strain equal to the FRP coupon rupture strain $\varepsilon_{F,2}$, the measured lower value of $\varepsilon_{F,1}$ at that instance represents the FRP strain at another location. They reported several measured FRP strains at the location of rupture in columns that were similar to coupon rupture strains. It is, however, accepted that some fibers would inherently be weaker than others and start reaching their ultimate stress and strain while others fibers are still healthy. The peak load carrying capacity of concrete would generally correspond to the start of the fiber rupture or soon after, but this will not represent the ultimate failure of the column. Progressive rupture of fibers beyond this point results in softening of concrete and provides the descending
part of its response. This was also verified by the gradual rupture of FRP strips observed in the current experiment. The progressive rupture of FRP also explains the more sudden and brittle failure of the small-scale FRP-confined columns compared with similarly confined larger-scale ones.

The higher efficiency coefficient $k_{e1}$ in this research could also be partly attributed to the strain gradient in the columns section under simulated seismic loading. Unlike the evenly tensile FRP stress in FRP-confined columns subjected to concentric compression, the largest FRP stress occurs at the most compressed face within the potential plastic-hinge regions and less at other locations for columns under seismic loading. Thus initial rupture of the individual most stressed FRP strips did not lead to immediate failure of the whole FRP wrapping and the capacity of columns could still be maintained.

The FRP rupture strains $\varepsilon_{F,2}$ obtained from the coupon tests were much lower than those to be expected from the dry fabric used to fabricate the FRP jackets, as shown in Table 3.2. The earlier rupture of fibers in the coupon tests is a well-known phenomenon. If the mechanical properties of dry fabric and epoxy resin are used in design, a much lower efficiency coefficient of FRP would result. This is shown by the coefficient $k_{e2}=\varepsilon_{F,1}/\varepsilon_{F,3}$ in Table 4.3, in which $\varepsilon_{F,3}$ is the ultimate strain of dry fabric reported by the manufacture. The average value of coefficient $k_{e2}$ is 0.633 for CFRP and 0.359 for GFRP jackets in the seven FRP-confined specimens in this experimental research.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>FRP wrapping</th>
<th>Maximum FRP strain $\varepsilon_{F,1}$ in column test</th>
<th>Maximum strain $\varepsilon_{F,2}$ in FRP coupon test</th>
<th>$k_{e1}=\frac{\varepsilon_{F,1}}{\varepsilon_{F,2}}$</th>
<th>$k_{e2}=\frac{\varepsilon_{F,1}}{\varepsilon_{F,3}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>P27-1CF-3</td>
<td>1-layer CFRP</td>
<td>0.01017</td>
<td>0.01229</td>
<td>0.828</td>
<td>0.598</td>
</tr>
<tr>
<td>P27-2GF-4</td>
<td>2-layer GFRP</td>
<td>0.01461</td>
<td>0.02031</td>
<td>0.719</td>
<td>0.325</td>
</tr>
<tr>
<td>P40-1CF-8</td>
<td>1-layer CFRP</td>
<td>0.01115</td>
<td>0.01229</td>
<td>0.907</td>
<td>0.656</td>
</tr>
<tr>
<td>P40-1GF-9</td>
<td>1-layer GFRP</td>
<td>0.01659</td>
<td>0.02031</td>
<td>0.817</td>
<td>0.369</td>
</tr>
<tr>
<td>P56-2CF-13</td>
<td>2-layer CFRP</td>
<td>0.01097</td>
<td>0.01229</td>
<td>0.893</td>
<td>0.645</td>
</tr>
<tr>
<td>P56-3GF-14</td>
<td>3-layer GFRP</td>
<td>0.01808</td>
<td>0.02031</td>
<td>0.890</td>
<td>0.402</td>
</tr>
<tr>
<td>P56-4GF-15</td>
<td>4-layer GFRP</td>
<td>0.01535</td>
<td>0.02031</td>
<td>0.756</td>
<td>0.341</td>
</tr>
</tbody>
</table>
4.4.6 Development of strains in FRP wrapping

Since FRP is nearly a linearly elastic material, the level of confinement is directly related to the strain in the FRP wrapping. The development of the FRP strain measured by strain gauge F1 is illustrated in Figure 4.20 for each of the seven FRP-confined columns. Recall that the arrangement and numbering of strain gauges on the FRP surfaces have been presented in Figure 3.9. This strain gauge was installed on the top surface of the column at a distance of 150 mm from the column-stub interface and was very close to the critical section of each FRP-confined specimen. Transverse strains in all the columns developed in a similar pattern, i.e. increasing with cyclic excursions without returning to zero until the rupture of the FRP.

Another important observation was made about the distribution of the FRP strain around the circumference of the critical section. In all the seven FRP-confined columns, the FRP strains were measured by strain gauges at intervals of 90° around the circumference of the critical section. An example is presented in Figure 4.21, in which the strains measured by four strain gauges F1, F2, F3 and F4 of specimen P56-4GF-15 are compared. It is found that a similar strain pattern was measured by these four strain gauges within each cycle of lateral excursion. The only difference was that the range of the strains in F1 and F3 was bigger than those in F2 and F4. This observation indicated that the concrete behaved almost like fluid to some extent during the plastic deformation. In the ultimate cycle of lateral load, the concrete within the critical region was found to have been crushed into disintegrated granules. Thus, the transverse confining stress in the FRP wrapping was almost uniform around the column circumference.

For columns subjected to seismic loading, the transverse strains around a cross section are usually deemed to be quite different due to the longitudinal strain gradient in the cross section of columns. This philosophy has been reflected in the conventional sectional analysis method, in which the stress-strain relationship of the FRP-confined concrete under uniform compression was used directly (Teng et al. 2002). It was implied that the transverse confining pressure in an eccentrically loaded column was non-uniform, varied from zero at the neutral position to a maximum at the extreme compressive fiber of the concrete. Accordingly, the strain in F1 should decrease to zero when the column top side goes into tension while it should be much larger than the strains in F2 and F3 when the column top side was in compression. Data from the tests did not appear to support this assertion.
Figure 4.20 FRP strain developments measured by gauge F1
4.4.7 Effectiveness of steel spirals

This topic is discussed in two different domains: eight steel-confined columns and seven FRP-confined columns in this experimental program.

1) Steel-confined columns

The maximum strains measured in steel spirals are presented in Table 4.4 for the eight steel-confined columns until the occurrence of the ultimate curvature $\phi_u$ in the plastic-hinge regions.
When closely spaced which satisfied the seismic design provisions of CAN/CSA A23.3-04 for ductile or moderately-ductile columns, the spirals displayed full effectiveness. Columns P27-NF-1, P27-NF-2, P27-NF-6, P27-NF-7, P27-NF-11, and P27-NF-12 belonged to this category. The failure of the columns was initially indicated by the buckling of longitudinal bars and finally occurred with the fracture of spirals in an explosive manner. Limited by the measuring capacity of the strain gauges, the actual ultimate strain of spirals could not be captured. But, all the maximum recorded spiral strains in this category were greater than 0.01, i.e. at least four times the yield strain of spirals. Thus the yield strength of the spirals can be used in estimating the maximum transverse confinement.

On the contrary, if the spacing of spirals was large such as that designed in accordance with most concrete codes prior to the 1970s, the steel spirals could hardly be fully effective. The control specimens P40-NF-5 and P56-NF-10 with only #3@300 mm spirals were the typical examples for this case. The measured results revealed that no spiral yielded in these two specimens during most of the test process until the emergence of severe buckling of longitudinal reinforcement. No spiral fracture was observed even after the final failure of columns. The confinement effect in columns with widely spaced transverse reinforcement is deemed negligible.

Table 4.4 Maximum strains of spirals in steel-confined specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Spirals</th>
<th>Maximum spiral strain $\varepsilon_m (\mu\varepsilon)$</th>
<th>Spiral yield strain $\varepsilon_y (\mu\varepsilon)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>P27-NF-1</td>
<td>#3@150</td>
<td>18100</td>
<td>2500</td>
</tr>
<tr>
<td>P27-NF-2</td>
<td>#3@100</td>
<td>10275</td>
<td>2500</td>
</tr>
<tr>
<td>P40-NF-5</td>
<td>#3@300</td>
<td>12493</td>
<td>2500</td>
</tr>
<tr>
<td>P40-NF-6</td>
<td>#3@100</td>
<td>11100</td>
<td>2500</td>
</tr>
<tr>
<td>P40-NF-7</td>
<td>#3@78</td>
<td>17200</td>
<td>2500</td>
</tr>
<tr>
<td>P56-NF-10</td>
<td>#3@300</td>
<td>3500</td>
<td>2500</td>
</tr>
<tr>
<td>P56-NF-11</td>
<td>10M@100</td>
<td>16700</td>
<td>2400</td>
</tr>
<tr>
<td>P56-NF-12</td>
<td>10M@77</td>
<td>13900</td>
<td>2400</td>
</tr>
</tbody>
</table>

2) FRP-retrofitted columns

The seven FRP-confined specimens had the same widely spaced spirals of #3@300 mm. The test data clearly indicated that the development of spiral strains lagged significantly behind the
corresponding FRP strains within the plastic-hinge regions. This is demonstrated by an example in Figure 4.22, in which the development of strain in FRP jackets and steel spirals at the critical section of column P56-2CF-13 is presented. The tensile strain in the FRP wrapping was always much greater than that in the spiral at the same cross section of column. The measured strains in the FRP jackets and spirals are compared in Table 4.5 for each FRP-confined specimen at the first peak point of the 4th lateral deflection cycle. At this moment, no spiral yielded while all the FRP strain values were greater than 2500 με, which is approximately the yield strain of the spiral steel. It can also be found that in all the FRP-confined specimens, the yield of spirals occurred much later than in the steel-confined counters, which has been indicated by the labels in the shear vs. tip deflection curves in Figure 4.3 to Figure 4.17.

Figure 4.22 Strain development in steel spirals and FRP jackets of P56-2CF-13

In the conventional sectional analysis, the strains in the FRP wrapping, transverse steel and concrete at the same cross section are assumed to be equal. Considering the data from seven specimens presented here, this does not appear to be a valid assumption. This phenomenon can be explained by the following two reasons. Firstly, concrete was seriously pulverized during the reversed severe plastic deformation, especially the cover concrete. Most of the granulated concrete tended to push directly on the continuous FRP jackets instead of the discrete steel spirals, which resulted in much larger strain in FRP wrapping than in spirals. Secondly, between the widely spaced spirals, the buckling longitudinal reinforcement pushed outwards onto the FRP jackets and led to the great increase of FRP strains. This observation indicated that the efficiency of widely spaced spirals was not reliable in such columns and had better be ignored in
confinement design. However, due to a lack of sufficient experimental data of columns transversely confined with a combination of FRP wrapping and closely spaced spirals, the reliability of this kind of steel spirals is still waiting for further validation.

Table 4.5 Strains in spirals and FRP jackets of FRP-confined specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Spirals</th>
<th>FRP Jackets</th>
<th>Yield strain of spirals, $\varepsilon (\mu\varepsilon)$</th>
<th>At the 1st peak of the 4th lateral deflection cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Maximum strain in spirals, $\varepsilon (\mu\varepsilon)$</td>
<td>Maximum strain in FRP, $\varepsilon_{FRP} (\mu\varepsilon)$</td>
</tr>
<tr>
<td>P27-1CF-3</td>
<td>#3@300</td>
<td>1-layer CFRP</td>
<td>2500</td>
<td>1979</td>
</tr>
<tr>
<td>P27-2GF-4</td>
<td>#3@300</td>
<td>2-layer GFRP</td>
<td>2500</td>
<td>383</td>
</tr>
<tr>
<td>P40-1CF-8</td>
<td>#3@300</td>
<td>1-layer CFRP</td>
<td>2500</td>
<td>1013</td>
</tr>
<tr>
<td>P40-1GF-9</td>
<td>#3@300</td>
<td>1-layer GFRP</td>
<td>2500</td>
<td>469</td>
</tr>
<tr>
<td>P56-2CF-13</td>
<td>#3@300</td>
<td>2-layer CFRP</td>
<td>2500</td>
<td>564</td>
</tr>
<tr>
<td>P56-3GF-14</td>
<td>#3@300</td>
<td>3-layer GFRP</td>
<td>2500</td>
<td>266</td>
</tr>
<tr>
<td>P56-4GF-15</td>
<td>#3@300</td>
<td>4-layer GFRP</td>
<td>2500</td>
<td>847</td>
</tr>
</tbody>
</table>

4.4.8 Deformability of FRP-confined concrete

The deformability of concrete is one of the most important aspects that can be significantly improved by transverse confinement. In the sectional analysis of columns under seismic loading, it is found that the curvature ductility of FRP-confined columns is mainly controlled by the ultimate compressive strain of the concrete in the longitudinal direction of columns.

The ultimate compressive strain $\varepsilon_{cu}$ of unconfined concrete is specified as 0.003 in ACI 318-11, while it is 0.0035 in CAN/CSA-A23.3-04. In this section the compressive strain of concrete is taken as positive. For the FRP-confined concrete, several methods have been proposed to evaluate the improved ultimate strain $\varepsilon_{cu}$, such as the following expression proposed by Lam and Teng (2003), referred to as the Lam-Teng model here.

$$\frac{\varepsilon_{cu}}{\varepsilon_{co}} = 1.75 + 12 \left( \frac{f_{tu}}{f_c} \right) \left( \frac{\varepsilon_{k,up}}{\varepsilon_c} \right)^{0.45}$$

where: $f_c = \text{compressive strength of unconfined concrete}; \ \varepsilon_c = \text{axial strain at the peak stress of}$
unconfined concrete (normally taken as 0.002); $f_{la}$ = actual maximum confining pressure at FRP rupture, taken as $\frac{2E_FT_F\varepsilon_{h,rup}}{D}$ for circular columns; $E_F$ and $t_F$ are the Young’s modulus and thickness of FRP jackets; $D$ = diameter of column; $\varepsilon_{h,rup}$ = actual rupture strain of FRP jackets. Based on test results, Lam and Teng (2004) reported that the value of $\varepsilon_{h,rup}$ was lower than the ultimate FRP strain $\varepsilon_{Fu}$ measured from the FRP flat coupon tests, and recommended $\varepsilon_{h,rup} = 0.637\varepsilon_{Fu}$ for CFRP and $\varepsilon_{h,rup} = 0.744\varepsilon_{Fu}$ for GFRP wrapping. The difference of predictions of $\varepsilon_{cu}$ was not very significant among most proposed models, since they were mostly based on similar experimental database.

However, most of these models were based on the experiments of small-scaled concrete cylinders subjected to concentric axial compression and were used directly in sectional analysis for columns subjected to seismic loading. It means that the effect of strain gradient in the cross section and the size effect are assumed to have negligible effects on the stress-strain relationship of FRP-confined concrete. This assumption is still waiting for validation, thus the ultimate concrete strain in this experimental program is investigated here.

For the seven FRP-confined concrete columns, the measured maximum strains, $\varepsilon_{cu,1}$, of the extreme compressive concrete fiber at the critical section are presented in Table 4.6. The value of $\varepsilon_{cu,1}$ was estimated from the deformation of concrete core measured by the LVDTs installed within the test region of columns. Because the LVDTs could hardly be held very firmly in the last stage of plastic deformation due to the serious damage of concrete, the values of $\varepsilon_{cu,1}$ presented in Table 4.6 represent the reliable records just before that last stage. The reported values are thus somewhat smaller than actually achieved maximum levels. In the same table, the ultimate strain of concrete calculated by using Eq. (4-8) is also shown and is denoted as $\varepsilon_{cu,2}$.

The comparison in Table 4.6 indicates that the predicted ultimate strains $\varepsilon_{cu,2}$ were much smaller than the measured $\varepsilon_{cu,1}$ in all seven FRP-confined specimens. The ratio of $\varepsilon_{cu,1}/\varepsilon_{cu,2}$ ranged from 1.34 to 6.13, with an average of 3.07. Moreover, the ratio $\varepsilon_{cu,1}/\varepsilon_{cu,2}$ for columns under a low axial loading level ($P/P_o = 0.27$) was much higher than that of columns subjected to a high axial load level ($P/P_o = 0.4$ or 0.56), as shown in Figure 4.23. For example, the specimen P27-1CF-3 was subjected to axial load of 0.27$P_o$ while P40-1CF-8 was tested under 0.40$P_o$ and both of them were confined with one layer of CFRP wrapping. The ratio of $\varepsilon_{cu,1}/\varepsilon_{cu,2}$ was 6.13 in the former
and 1.60 in the latter. Thus, the ultimate compressive strain of concrete, $\varepsilon_{cu,1}$ was markedly influenced by the strain gradient in the cross section. While Eq. (4-8) may be valid for FRP-confined concrete under concentric compression, its use for the sectional analysis of the FRP-confined concrete columns under eccentric compression will result in very conservative estimate of curvature ductility.

Table 4.6 Maximum compressive strains in FRP-confined concrete

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Axial load $P/P_o$</th>
<th>Measured $\varepsilon_{cu,1}$</th>
<th>Predicted $\varepsilon_{cu,2}$</th>
<th>Ratio of $\varepsilon_{cu,1} / \varepsilon_{cu,2}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>P27-1CF-3</td>
<td>0.27</td>
<td>0.0443</td>
<td>0.00723</td>
<td>6.13</td>
</tr>
<tr>
<td>P27-2GF-4</td>
<td>0.27</td>
<td>0.0625</td>
<td>0.01157</td>
<td>5.40</td>
</tr>
<tr>
<td>P40-1CF-8</td>
<td>0.40</td>
<td>0.0116</td>
<td>0.00723</td>
<td>1.60</td>
</tr>
<tr>
<td>P40-1GF-9</td>
<td>0.40</td>
<td>0.0259</td>
<td>0.00753</td>
<td>3.44</td>
</tr>
<tr>
<td>P56-2CF-13</td>
<td>0.56</td>
<td>0.0206</td>
<td>0.01095</td>
<td>1.88</td>
</tr>
<tr>
<td>P56-3GF-14</td>
<td>0.56</td>
<td>0.0209</td>
<td>0.01560</td>
<td>1.34</td>
</tr>
<tr>
<td>P56-4GF-15</td>
<td>0.56</td>
<td>0.0328</td>
<td>0.01964</td>
<td>1.67</td>
</tr>
</tbody>
</table>

Figure 4.23 Relationships between ratio of ultimate concrete strain and axial load
4.4.9 Equivalent length of plastic hinge

During major earthquakes, severe damage and large plastic curvature may concentrate in regions with large moment in the concrete columns, which are referred to as plastic hinges. The length of plastic hinge is an important parameter in this research, connecting the sectional ductility behaviour with the element behaviour.

As proposed by Park and Paulay (1975) for concrete columns, the equivalent length of plastic hinge, \( L_p \), was the length over which the plastic curvature was assumed to be constant. Following relationship was derived for a cantilever column:

\[
\Delta_p = \Delta_{\text{max}} - \Delta_y = (\phi_{\text{max}} - \phi_y) L_p (L - 0.5L_p) \tag{4-9}
\]

where \( \Delta_y \) and \( \phi_y \) are the yield displacement and curvature, respectively; \( \Delta_{\text{max}} \) and \( \phi_{\text{max}} \) are the measured values of the maximum displacement and curvature in each lateral excursion cycle during testing, respectively; and \( L \) is the cantilever length of column. Using this equation, \( L_p \) of each specimen was estimated in each lateral excursion cycle in which \( \mu \Delta \) is greater than 2.

Note that the estimated \( L_p \) by Eq. (4-9) includes the effects of yield penetration of longitudinal reinforcement into the stub and any diagonal tension cracking. It is assumed that the plastic hinge is located right adjacent to the column-stub interface. Even though the plastic hinge is shifted away from the stub face for a distance due to the additional confinement by the heavily reinforced stub, this offset distance is small compared to \( L \) and thus neglected in this equation.

It was reported by Sheikh and Khoury (1997) and Sheikh and Yau (2002) that \( L_p \) is approximately equal to the cross section size, \( D \), of the confined concrete columns. This implied the independence of the plastic hinge length to the cross section shape, the steel configuration, level of axial load and the amount of transverse confinement. Some researchers such as Bae and Bayrak (2008), however, argued that the plastic hinge length of the column was significantly affected by the aspect ratio \( L/D \) and the axial load level of the column. They reported that the plastic hinge length increased with \( L/D \) and \( P/P_o \).

The average value of \( L_p \) of each column is estimated by using Eq. (4-9) and presented in Table 4.7 for specimens in this experimental program and those reported by Sheikh and Yau (2002). However, the control specimens with only #3@300 mm spirals are not included, because they all
failed in a very brittle manner without sufficient development of the plastic hinges. For comparison, the actual measured length of the most damaged region, $L_{dr}$, is also presented in Table 4.7 for each specimen. It is found that the estimated equivalent plastic hinge length $L_p$ is similar to the observation of $L_{dr}$. These $L_p$ values are approximately equal to the diameter of columns, $D$, and do not show clear dependence on the steel configuration, axial load level, and the type and amount of transverse confinement, as indicated in Figure 4.24.

### Table 4.7 Equivalent plastic hinge length

<table>
<thead>
<tr>
<th>Researcher</th>
<th>Specimen</th>
<th>Number of cycles</th>
<th>$\mu_\Delta$</th>
<th>Axial load $P/P_o$</th>
<th>Equivalent plastic hinge length, $L_p$</th>
<th>Length of most damaged region, $L_{dr}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\text{Average } L_p$ (mm)</td>
<td>$L_p/D$</td>
</tr>
<tr>
<td>Current tests</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P27-NF-1</td>
<td>6</td>
<td>3.4</td>
<td>0.27</td>
<td></td>
<td>334</td>
<td>0.94</td>
</tr>
<tr>
<td>P27-NF-2</td>
<td>9</td>
<td>3.8</td>
<td>0.27</td>
<td></td>
<td>379</td>
<td>1.06</td>
</tr>
<tr>
<td>P27-1CF-3</td>
<td>12</td>
<td>5.2</td>
<td>0.27</td>
<td></td>
<td>320</td>
<td>0.90</td>
</tr>
<tr>
<td>P27-2GF-4</td>
<td>16</td>
<td>4.9</td>
<td>0.27</td>
<td></td>
<td>293</td>
<td>0.82</td>
</tr>
<tr>
<td>P40-NF-5</td>
<td>3</td>
<td>2.2</td>
<td>0.40</td>
<td></td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>P40-NF-6</td>
<td>6</td>
<td>3.2</td>
<td>0.40</td>
<td></td>
<td>268</td>
<td>0.75</td>
</tr>
<tr>
<td>P40-NF-7</td>
<td>7</td>
<td>4.0</td>
<td>0.40</td>
<td></td>
<td>439</td>
<td>1.23</td>
</tr>
<tr>
<td>P40-1CF-8</td>
<td>7</td>
<td>6.9</td>
<td>0.40</td>
<td></td>
<td>242</td>
<td>0.68</td>
</tr>
<tr>
<td>P40-1GF-9</td>
<td>7</td>
<td>6.3</td>
<td>0.40</td>
<td></td>
<td>276</td>
<td>0.78</td>
</tr>
<tr>
<td>P56-NF-10</td>
<td>3</td>
<td>2.1</td>
<td>0.56</td>
<td></td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>P56-NF-11</td>
<td>6</td>
<td>3.3</td>
<td>0.56</td>
<td></td>
<td>238</td>
<td>0.69</td>
</tr>
<tr>
<td>P56-NF-12</td>
<td>7</td>
<td>3.7</td>
<td>0.56</td>
<td></td>
<td>297</td>
<td>0.83</td>
</tr>
<tr>
<td>P56-2CF-13</td>
<td>10</td>
<td>7.5</td>
<td>0.56</td>
<td></td>
<td>327</td>
<td>0.92</td>
</tr>
<tr>
<td>P56-3GF-14</td>
<td>14</td>
<td>7.2</td>
<td>0.56</td>
<td></td>
<td>429</td>
<td>1.20</td>
</tr>
<tr>
<td>P56-4GF-15</td>
<td>17</td>
<td>8.8</td>
<td>0.56</td>
<td></td>
<td>232</td>
<td>0.65</td>
</tr>
<tr>
<td>Sheikh and Yau (2002)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S-1NT</td>
<td>8</td>
<td>2.0</td>
<td>0.50</td>
<td></td>
<td>422</td>
<td>1.19</td>
</tr>
<tr>
<td>S-2NT</td>
<td>12</td>
<td>2.7</td>
<td>0.25</td>
<td></td>
<td>315</td>
<td>0.88</td>
</tr>
<tr>
<td>S-3NT</td>
<td>2</td>
<td>2.4</td>
<td>0.51</td>
<td></td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>S-4NT</td>
<td>5</td>
<td>2.1</td>
<td>0.25</td>
<td></td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>ST-2NT</td>
<td>11</td>
<td>5.5</td>
<td>0.50</td>
<td></td>
<td>438</td>
<td>1.23</td>
</tr>
<tr>
<td>ST-3NT</td>
<td>11</td>
<td>3.9</td>
<td>0.50</td>
<td></td>
<td>400</td>
<td>1.12</td>
</tr>
<tr>
<td>ST-4NT</td>
<td>15</td>
<td>4.3</td>
<td>0.23</td>
<td></td>
<td>550</td>
<td>1.55</td>
</tr>
<tr>
<td>ST-5NT</td>
<td>15</td>
<td>4.1</td>
<td>0.25</td>
<td></td>
<td>452</td>
<td>1.27</td>
</tr>
</tbody>
</table>
4.4.10 Drift ratio of columns

The lateral drift ratio of the cantilever column is defined as $\delta = \Delta_u / L$, where: $\Delta_u =$ ultimate lateral tip deflection at the first occurrence of a 20% decay of lateral shear resistance $V$ or the collapse of column; $L =$ cantilever length of column, measured from the stub-column interface to the contraflexure point, which is also the shear span of the column. The definitions of $\Delta_u$ and $L$ are the same as in the previous sections. The experimental results of drift ratio $\delta$ are presented in Table 4.8 and Figure 4.25 for the specimens in this program, as well as those reported by Sheikh and Yau (2002).

The test data indicates that the lateral drift capacity increased with the confinement index $I_e = f_i / f_c$ for specimens subjected to the similar axial load level. But it is found the drift capacity $\delta$ was significantly influenced by the $P$-$\Delta$ effect. Seven out of the eight steel-confined columns which were subjected to high axial load levels ($P/P_o \geq 0.4$) got drift ratios lower than 2.2%. For example, the specimens P56-NF-12 and S-1NT were transversely confined by closely spaced steel spirals and tested under high axial load of $0.56P_o$ and $0.50P_o$, respectively. Both columns could just achieve a similar drift ratio of about 2.15%. On the contrary, tested under $P = 0.27P_o$ and confined solely by widely spaced spirals of #3@300 mm, the control specimen S-4NT could gain a drift ratio of 3.0%. Moreover, for P27-NF-2 with closely spaced spiral of #3@100 mm and subjected to $P = 0.27P_o$, the drift ratio was up to 3.54%. This indicated that the drift capacity of the column was limited by the severe $P$-$\Delta$ effect caused by the high axial load level.
<table>
<thead>
<tr>
<th>Researcher</th>
<th>Specimen</th>
<th>Transverse confinement</th>
<th>Axial load level, $P/P_o$</th>
<th>Ultimate tip deflection, $\Delta_u$ (mm)</th>
<th>Drift ratio, $\delta$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P27-NF-1</td>
<td>#3@150</td>
<td>--</td>
<td>0.031</td>
<td>0.27</td>
<td>58.5</td>
</tr>
<tr>
<td>P27-NF-2</td>
<td>#3@100</td>
<td>--</td>
<td>0.051</td>
<td>0.27</td>
<td>65.1</td>
</tr>
<tr>
<td>P27-1CF-3</td>
<td>#3@300</td>
<td>1-layer CFRP</td>
<td>0.109</td>
<td>0.27</td>
<td>87.9</td>
</tr>
<tr>
<td>P27-2GF-4</td>
<td>#3@300</td>
<td>2-layer GFRP</td>
<td>0.131</td>
<td>0.27</td>
<td>83.0</td>
</tr>
<tr>
<td>P40-NF-5</td>
<td>#3@300</td>
<td>--</td>
<td>--</td>
<td>0.40</td>
<td>30.6</td>
</tr>
<tr>
<td>P40-NF-6</td>
<td>#3@100</td>
<td>--</td>
<td>0.051</td>
<td>0.40</td>
<td>40.1</td>
</tr>
<tr>
<td>P40-NF-7</td>
<td>#3@75</td>
<td>--</td>
<td>0.072</td>
<td>0.40</td>
<td>57.3</td>
</tr>
<tr>
<td>P40-1CF-8</td>
<td>#3@300</td>
<td>1-layer CFRP</td>
<td>0.120</td>
<td>0.40</td>
<td>78.5</td>
</tr>
<tr>
<td>P40-1GF-9</td>
<td>#3@300</td>
<td>1-layer GFRP</td>
<td>0.074</td>
<td>0.40</td>
<td>70.8</td>
</tr>
<tr>
<td>P56-NF-10</td>
<td>#3@300</td>
<td>--</td>
<td>--</td>
<td>0.56</td>
<td>23.0</td>
</tr>
<tr>
<td>P56-NF-11</td>
<td>10M@100</td>
<td>--</td>
<td>0.066</td>
<td>0.56</td>
<td>40.9</td>
</tr>
<tr>
<td>P56-NF-12</td>
<td>10M@75</td>
<td>--</td>
<td>0.092</td>
<td>0.56</td>
<td>39.2</td>
</tr>
<tr>
<td>P56-2CF-13</td>
<td>#3@300</td>
<td>2-layer CFRP</td>
<td>0.236</td>
<td>0.56</td>
<td>89.8</td>
</tr>
<tr>
<td>P56-3GF-14</td>
<td>#3@300</td>
<td>3-layer GFRP</td>
<td>0.243</td>
<td>0.56</td>
<td>86.1</td>
</tr>
<tr>
<td>P56-4GF-15</td>
<td>#3@300</td>
<td>4-layer GFRP</td>
<td>0.275</td>
<td>0.56</td>
<td>92.9</td>
</tr>
<tr>
<td>S-1NT</td>
<td>#3@80</td>
<td>--</td>
<td>0.068</td>
<td>0.50</td>
<td>39.7</td>
</tr>
<tr>
<td>S-2NT</td>
<td>#3@80</td>
<td>--</td>
<td>0.068</td>
<td>0.25</td>
<td>74.9</td>
</tr>
<tr>
<td>S-3NT</td>
<td>#3@300</td>
<td>--</td>
<td>--</td>
<td>0.51</td>
<td>36.2</td>
</tr>
<tr>
<td>S-4NT</td>
<td>#3@300</td>
<td>--</td>
<td>--</td>
<td>0.25</td>
<td>55.2</td>
</tr>
<tr>
<td>ST-2NT</td>
<td>#3@300</td>
<td>2-layer GFRP</td>
<td>0.107</td>
<td>0.50</td>
<td>79.0</td>
</tr>
<tr>
<td>ST-3NT</td>
<td>#3@300</td>
<td>1-layer CFRP</td>
<td>0.081</td>
<td>0.50</td>
<td>77.5</td>
</tr>
<tr>
<td>ST-4NT</td>
<td>#3@300</td>
<td>1-layer CFRP</td>
<td>0.073</td>
<td>0.23</td>
<td>105.1</td>
</tr>
<tr>
<td>ST-5NT</td>
<td>#3@300</td>
<td>1-layer GFRP</td>
<td>0.053</td>
<td>0.25</td>
<td>100.9</td>
</tr>
</tbody>
</table>

The experimental results showed that FRP-confinement had much higher efficiency than the conventional steel confinement to improve the lateral drift capacity of columns. As presented in Table 4.8, except specimen S-2NT, none of the twelve steel-confined columns can achieve a drift capacity of 4.0%. On the contrary, ten out of the eleven FRP-retrofitted columns had drift capacity higher than 4.0%. The average value of drift capacity was 4.70% for the eleven FRP-retrofitted specimens while it was only 2.82% for the eight columns confined by conventional closely spaced steel spirals. The superiority of FRP confinement in improving drift capacity $\delta$...
was mainly because of two reasons: First, with the entire cross section confined by FRP jackets, the strength loss of columns due to spalling of concrete cover was avoided until the final failure; Second, the confining pressure by FRP jackets kept increasing with the development of plastic deformation and the dilation of column cross section. Thus, the moment vs. curvature \((M-\phi)\) envelopes of FRP-retrofitted specimens showed almost no strength decline until the final failure.

![Figure 4.25 Relationships between drift ratio and confinement](image)

The lateral drift ratio is not affected by the various definitions and measurements of the yield displacement and yield curvature, therefore it is easy to be measured and reported in the published literature. It should be noted that the drift ratio represents the overall lateral deflection capacity of the column but does not separate the plastic deformation from the elastic deformation. Thus, drift ratio can only be used to reflect the overall lateral deformability and stiffness of the columns rather than their ductility factors.

### 4.4.11 Flexural strength enhancement

The test results indicated that the flexural strengths of columns might be enhanced by transverse confinement significantly, as shown in Table 4.9. The enhancement of the flexural strength is represented by the ratio \(M_{\text{max}}/M_n\), in which \(M_{\text{max}}\) is the measured flexural strength and \(M_n\) is the nominal flexural strength of the concrete column. The definition of \(M_n\) was introduced
previously in Section 4.3.2 and similar to the amount of flexural strength estimated according to ACI 318-11.

A number of issues related to the flexural strength enhancement are discussed here. Firstly, the test results showed that the transverse FRP confinement was more efficient to provide strength enhancement than steel confinement. The average value of $M_{\text{max}}/M_n$ ratio was 1.46 for the eleven FRP-retrofitted specimens and 1.05 for the other eight columns confined by closely spaced steel spirals. The moment vs. curvature ($M$-$\phi$) envelopes of the eleven FRP-retrofitted columns showed almost no strength degradation until final failure, because the whole cross section was confined by the FRP jackets without concrete cover spalling. However, in the eight steel-confined columns, the spalling of the concrete cover led to a significant loss of section area and therefore the descending post-peak branch of $M$-$\phi$ envelopes.

Secondly, it has also been noted that in the tests reported by Watson and Park (1994), the flexural strength enhancement $M_{\text{max}}/M_n$ of the steel-confined columns was measured from 1.1 to 2.0, which was much higher than those measured in the current experimental program. To estimate the flexural strength enhancement of columns due to steel confinement, the following two empirical expressions were suggested by Watson and Park (1994):

\[
\frac{M_{\text{max}}}{M_n} = 1.13 \quad \text{(for } \frac{P}{f_c A_g} \leq 0.1) \quad (4-10)
\]

\[
\frac{M_{\text{max}}}{M_n} = 1.13 + 2.35 \left( \frac{P}{f_c A_g} - 0.1 \right)^{2.0} \quad \text{(for } \frac{P}{f_c A_g} > 0.1) \quad (4-11)
\]

These two expressions were found to be invalid for the current experimental program. Close study revealed that the clear concrete cover was only 13 mm in the 400×400 mm square columns tested by Watson and Park (1994) and thus the ratio of core concrete to the gross section area ($A_c/A_g$) was about 84%. In the current experimental program, the ratio of $A_c/A_g$ was about 74%. This indicated that the flexural strength enhancement of steel-confined columns is significantly affected by the amount of $A_c/A_g$.

Thirdly, the enhancement of flexural strength increased almost proportionally with the transverse confinement level, as indicated clearly by the results of FRP-confined columns in Figure 4.26.
<table>
<thead>
<tr>
<th>Researcher Specimen</th>
<th>Transverse confinement</th>
<th>Axial load level $P/P_o$</th>
<th>$M_{max}$ (kNm)</th>
<th>$M_n$ (kNm)</th>
<th>$M_{max}/M_n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>P27-NF-1 #3@150</td>
<td>0.031</td>
<td>0.27</td>
<td>204</td>
<td>217</td>
<td>0.940</td>
</tr>
<tr>
<td>P27-NF-2 #3@100</td>
<td>0.051</td>
<td>0.27</td>
<td>220</td>
<td>217</td>
<td>1.014</td>
</tr>
<tr>
<td>P27-1CF-3 1-layer CFRP 0.109</td>
<td>0.27</td>
<td>264</td>
<td>217</td>
<td>1.217</td>
<td></td>
</tr>
<tr>
<td>P27-2GF-4 2-layer GFRP 0.131</td>
<td>0.27</td>
<td>251</td>
<td>217</td>
<td>1.157</td>
<td></td>
</tr>
<tr>
<td>P40-NF-5 #3@300</td>
<td>--</td>
<td>0.40</td>
<td>180</td>
<td>204</td>
<td>0.882</td>
</tr>
<tr>
<td>P40-NF-6 #3@100</td>
<td>0.051</td>
<td>0.40</td>
<td>205</td>
<td>204</td>
<td>1.005</td>
</tr>
<tr>
<td>P40-NF-7 #3@75</td>
<td>0.072</td>
<td>0.40</td>
<td>229</td>
<td>204</td>
<td>1.123</td>
</tr>
<tr>
<td>P40-1CF-8 1-layer CFRP 0.120</td>
<td>0.40</td>
<td>261</td>
<td>204</td>
<td>1.279</td>
<td></td>
</tr>
<tr>
<td>P40-1GF-9 1-layer GFRP 0.074</td>
<td>0.40</td>
<td>258</td>
<td>204</td>
<td>1.265</td>
<td></td>
</tr>
<tr>
<td>P56-NF-10 #3@300</td>
<td>--</td>
<td>0.56</td>
<td>188</td>
<td>180</td>
<td>1.044</td>
</tr>
<tr>
<td>P56-NF-11 10M@100</td>
<td>0.066</td>
<td>0.56</td>
<td>203</td>
<td>180</td>
<td>1.128</td>
</tr>
<tr>
<td>P56-NF-12 10M@175</td>
<td>0.092</td>
<td>0.56</td>
<td>197</td>
<td>180</td>
<td>1.094</td>
</tr>
<tr>
<td>P56-2CF-13 2-layer CFRP 0.236</td>
<td>0.56</td>
<td>331</td>
<td>180</td>
<td>1.839</td>
<td></td>
</tr>
<tr>
<td>P56-3GF-14 3-layer GFRP 0.243</td>
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<td>180</td>
<td>1.872</td>
<td></td>
</tr>
<tr>
<td>P56-4GF-15 4-layer GFRP 0.275</td>
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<td>364</td>
<td>180</td>
<td>2.022</td>
<td></td>
</tr>
<tr>
<td>S-1NT #3@80</td>
<td>0.068</td>
<td>0.50</td>
<td>192</td>
<td>183</td>
<td>1.049</td>
</tr>
<tr>
<td>S-2NT #3@80</td>
<td>0.068</td>
<td>0.25</td>
<td>212</td>
<td>216</td>
<td>0.981</td>
</tr>
<tr>
<td>S-3NT #3@300</td>
<td>--</td>
<td>0.51</td>
<td>212</td>
<td>181</td>
<td>1.171</td>
</tr>
<tr>
<td>S-4NT #3@300</td>
<td>--</td>
<td>0.25</td>
<td>215</td>
<td>214</td>
<td>1.005</td>
</tr>
<tr>
<td>ST-2NT 2-layer GFRP 0.107</td>
<td>0.50</td>
<td>283</td>
<td>184</td>
<td>1.538</td>
<td></td>
</tr>
<tr>
<td>ST-3NT 1-layer CFRP 0.081</td>
<td>0.50</td>
<td>279</td>
<td>184</td>
<td>1.516</td>
<td></td>
</tr>
<tr>
<td>ST-4NT 1-layer CFRP 0.073</td>
<td>0.23</td>
<td>259</td>
<td>227</td>
<td>1.141</td>
<td></td>
</tr>
<tr>
<td>ST-5NT 1-layer GFRP 0.053</td>
<td>0.25</td>
<td>256</td>
<td>218</td>
<td>1.174</td>
<td></td>
</tr>
</tbody>
</table>
Lastly, it is observed that the enhancement of flexural strength, $M_{\text{max}}/M_n$, was larger in the columns subjected to higher axial load level $P/P_o$. For example, the specimens P27-2GF-4 and ST-2NT were identical in all respects except the level of axial load. The values of $M_{\text{max}}/M_n$ increased from 1.157 to 1.538 with the doubling of axial load level. The relationship between the value of $M_{\text{max}}/M_n$ and the corresponding axial load level $P/P_o$ is shown in Figure 4.27.

Based on the experimental results of the twelve steel-confined columns shown in Table 4.9,
together with the test data previously reported by Sheikh and Khoury (1993); Sheikh, et al. (1994); Waston and Park (1994); Saatcioglu and Baingo (1999); Legeron and Paultre (2000); Hwang and Yun, (2004); Bae (2005); and Paultre, et al. (2001, 2009), following empirical expression is proposed to estimate the flexural strength enhancement $M_{\text{max}}/M_n$ for steel-confined circular and square columns:

$$\frac{M_{\text{max}}}{M_n} = 1 + \frac{2.5}{\left(\frac{A_g}{A_c} - 1\right)} \left(\frac{f_l}{f'_c}\right) \left(\frac{P}{P_o}\right)^2$$  \hspace{1cm} (4-12)

The flexural strength enhancement of circular and square columns due to FRP-confinement is also studied based on experimental results. The test database of FRP-confined columns included the seven FRP-retrofitted circular columns in this experimental program, four 356 mm circular columns reported by Sheikh and Yau (2002), four 270 mm circular high-strength concrete columns by Ozbakkaloglu and Saatcioglu (2006), together with the thirteen 305×305 mm (12×12 in.) square columns by Iacobucci et al. (2003) and Memon and Sheikh (2005), as listed in Table 4.10.

The level of FRP-confinement is indicated in Table 4.10 by the ratio of nominal transverse confinement pressure in concrete, $f_{l,n}$, to the compressive strength of unconfined concrete, $f'_c$. Because the measured rupture strain of FRP wrapping during column testing was unavailable in most experimental reports of this database, the confinement is uniformly indicated by the nominal transverse confinement pressure or the so called theoretical maximum confining stress in concrete, $f_{l,n}$, which is determined based on the tensile strength, $f_{Fu}$, of FRP jackets from the FRP coupon tests by:

$$f_{l,n} = k_c \cdot \frac{2n_{F}t_{F}f_{Fu}}{D}$$  \hspace{1cm} (4-13)

where, $n_{F}$ and $t_{F}$ are the number of layers and thickness of per layer of FRP jackets, respectively; $D = $ diameter of circular columns or diagonal length of rectangular cross section of columns; $k_c = $ geometric efficiency coefficient of FRP jackets, which is 1.0 for columns with circular cross section and 0.452 for square columns listed in Table 4.10, in accordance with the proposition by ACI 440.2R-08 as introduced in Section 2.2.2.3.
Based on the test data in Table 4.10, following empirical expression is proposed to estimate the flexural strength enhancement $M_{max}/M_n$ for FRP-confined circular and square columns:

$$\frac{M_{max}}{M_n} = 1 + 8 \left( \frac{f_{lc}}{f_c} \right) \left( \frac{P}{P_o} \right)^{1.5}$$  \hspace{1cm} (4-14)

The validation of the expressions (4-12) and (4-14) is illustrated in Figure 4.28 for all the above mentioned specimens with good agreement. For the steel-confined columns showed in Figure 4.28 (a), the ratios of the predicted to experimental values of $M_{max}/M_n$ have an average of 1.003, with a standard deviation of 0.056. For the FRP-confined columns in Figure 4.28 (b), the ratios of the predicted to experimental values of $M_{max}/M_n$ have an average of 1.014, with a standard deviation of 0.061.

![Graph of Predictions of flexural strength enhancement](image)

Figure 4.28 Predictions of flexural strength enhancement

The strength enhancement of columns due to transverse confinement is still neglected by most design codes currently, while it is normally thought that transverse confinement can improve the flexural ductility without remarkable enhancement of the flexural strength of columns (CAN/CSA-S806-12). The experimental results revealed that it is necessary to recognize the enhancement of flexural strength in the confined region of columns, especially in the cases of FRP-retrofit.
### Table 4.10 Flexural strength enhancement in FRP-confined columns

<table>
<thead>
<tr>
<th>Researcher</th>
<th>Specimen</th>
<th>Transverse confinement</th>
<th>Axial load level</th>
<th>$M_{max}$ (kNm)</th>
<th>$M_n$ (kNm)</th>
<th>$\frac{M_{max}}{M_n}$</th>
</tr>
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<tr>
<td></td>
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<td>FRP jackets</td>
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<tr>
<td><strong>Current program</strong></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>P27-1CF-3</td>
<td>1-layer CFRP</td>
<td>0.132</td>
<td>0.27</td>
<td>264</td>
<td>217</td>
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<td>217</td>
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<td>--</td>
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<td>364</td>
<td>180</td>
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<tr>
<td><strong>Sheikh and Yau (2002)</strong></td>
<td>S-3NT</td>
<td>No FRP</td>
<td>--</td>
<td>0.51</td>
<td>212</td>
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<tr>
<td></td>
<td>S-4NT</td>
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<td>214</td>
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<tr>
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<td>ST-5NT</td>
<td>1-layer GFRP</td>
<td>0.071</td>
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<td>256</td>
<td>218</td>
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<tr>
<td><strong>Ozbakkaloglu and Saatcioglu (2006)</strong></td>
<td>RC-1</td>
<td>4-layer CFRP</td>
<td>0.206</td>
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<td>RC-4</td>
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<td>0.124</td>
<td>0.34</td>
<td>141</td>
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<td><strong>Iacobucci, et al. (2003)</strong></td>
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<td>-</td>
<td>0.33</td>
<td>181</td>
<td>200</td>
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<tr>
<td></td>
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<td>1-layer CFRP</td>
<td>0.055</td>
<td>0.33</td>
<td>229</td>
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</tr>
<tr>
<td></td>
<td>ASC-3NS</td>
<td>2-layer CFRP</td>
<td>0.109</td>
<td>0.56</td>
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</tr>
<tr>
<td></td>
<td>ASC-4NS</td>
<td>1-layer CFRP</td>
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<td>0.56</td>
<td>217</td>
<td>177</td>
</tr>
<tr>
<td></td>
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<td>0.163</td>
<td>0.56</td>
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<td></td>
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<td>2-layer CFRP</td>
<td>0.109</td>
<td>0.33</td>
<td>244</td>
<td>216</td>
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<tr>
<td></td>
<td>AS-8NS</td>
<td>No FRP</td>
<td>-</td>
<td>0.56</td>
<td>168</td>
<td>190</td>
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<tr>
<td><strong>Memon and Sheikh (2005)</strong></td>
<td>AS-1NSS</td>
<td>No FRP</td>
<td>-</td>
<td>0.57</td>
<td>168</td>
<td>182</td>
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<tr>
<td></td>
<td>ASG-2NSS</td>
<td>2-layer GFRP</td>
<td>0.055</td>
<td>0.34</td>
<td>226</td>
<td>215</td>
</tr>
<tr>
<td></td>
<td>ASG-3NSS</td>
<td>4-layer GFRP</td>
<td>0.110</td>
<td>0.57</td>
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<td>0.054</td>
<td>0.56</td>
<td>219</td>
<td>186</td>
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<td>1-layer GFRP</td>
<td>0.027</td>
<td>0.33</td>
<td>223</td>
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<td>6-layer GFRP</td>
<td>0.160</td>
<td>0.55</td>
<td>265</td>
<td>189</td>
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</table>
Chapter 5
Nonlinear Analysis of Columns

5.1 General

In order to study the seismic behaviour of confined concrete columns, extensive efforts have been made in experimental research on large-scale specimens in the past several decades (e.g. Sheikh and Khoury, 1993; Watson and Park, 1994; Bayrak and Sheikh, 1998; Sheikh and Yau, 2002; Bae, 2005; Ozbakkaloglu and Saatcioglu, 2006, 2007; Paultre et al., 2009). These columns were transversely confined by steel spirals or rectilinear ties or FRP jackets, and tested under constant axial load and lateral cyclic load. Despite a large number of reported tests, it is impossible to cover all the possible field conditions due to the difficulty of testing operation and considerable expenses. In addition, the test setup, loading history and definitions of ductility parameters in the published work differ from one program to another, making it difficult to evaluate the test results on a common platform. Available test data is still limited to develop and verify design procedures with confidence. Consequently, it is imperative to develop reliable analytical procedures which can predict the seismic behaviour of confined concrete columns with reasonable accuracy.

A nonlinear analytical procedure was reported by Razvi and Saatcioglu (1999) to conduct pushover analysis for steel-confined concrete columns. The main parameters considered were concrete confinement, anchorage slip and buckling of longitudinal steel bars, as well as the development of plastic hinge length. Based on this analysis, a displacement-based design procedure for steel-confined columns was proposed by using the lateral drift ratio as the criterion for seismic performance. However, a study of curvature ductility and displacement ductility of columns was not included in that analysis. Paultre and Légeron (2008), based on sectional analysis, also proposed a design procedure to determine the required confinement in steel-confined concrete columns. Only curvature ductility factor of columns was studied in this analysis and the seismic behaviour of columns measured by displacement ductility and the lateral drift ratio was not considered. Few analytical studies have been reported in the literature that study detailed seismic behaviour of steel-confined as well as FRP-confined columns within a consistent framework.
In this chapter a nonlinear analysis program is introduced, which is developed in the programming language Fortran for the monotonic pushover analysis to concrete columns. Named as SAC (Seismic Analysis of Columns), this computation program consists of two parts: the sectional analysis and the lateral pushover analysis. It provides response of columns in the form of envelope curves of the moment vs. curvature ($M-\phi$) and lateral shear vs. tip deflection ($V-\Delta$) hysteresis loops of cantilever columns subjected to simulated seismic loads. Calibrated and verified by existing test results with reasonable accuracy, it is applicable to square and circular columns with concrete strength ranging from 20 MPa to 120 MPa, transversely confined by steel reinforcement or FRP jackets. In the next chapter, it will be used to conduct extensive numerical analyses, based on which the seismic performance of confined concrete columns is studied.

The program SAC is aimed at the concrete columns of which the seismic performance is mainly controlled by flexural behaviour instead of shear. The shear span-depth ratio is not less than 3.0 and sufficient shear resistance of columns is presumed, since the brittle shear failure is beyond the scope of the current research.

5.2 Sectional analysis

5.2.1 Introduction

Sectional analysis is the primary step to investigate the seismic performance of a concrete column, in which the full range of the moment vs. curvature relationship is generated for the cross section. The laminar method is used for this analysis in which the cross section of column is divided into a series of laminas parallel to the neutral axis. The key steps of this analysis and the adopted constitutive relationships of concrete and reinforcement are introduced in the following sections.

5.2.2 Analysis procedure

The compatibility and equilibrium conditions in the sectional analysis are commonly available in the literature (e.g. Collins and Mitchell, 1991), therefore the details are not elaborated here. The computation process is briefly introduced as follows.
Under a certain axial load, the full range of the moment vs. curvature relationship is generated for a cross section by specifying a sequence of suitable values of strain $\varepsilon_{c,max}$ at the extreme compression fiber of concrete up to its ultimate value $\varepsilon_{cu}$. The stress and strain over the column depth are illustrated in Figure 5.1. For each value of $\varepsilon_{c,max}$, the location of the neutral axis is found by varying the depth of compression zone, $x$, iteratively until the resultant internal axial force $N$ calculated from Eq. (5-1) equals the externally applied axial load $P$. Based on the determined neutral axis position, the moment and corresponding curvature are determined by Eqs. (5-2) and (5-3), respectively. The flow chart of computation is shown in Figure 5.2.

$$N = \int_{\frac{d}{2}-x}^{\frac{d}{2}} \sigma_c b_c dy + \sum_{i=1}^{n} [(\sigma_{si} - \sigma_c)A_{si}]$$  \hspace{1cm} (5-1)

$$M = \int_{\frac{d}{2}-x}^{\frac{d}{2}} \sigma_c b_c y dy + \sum_{i=1}^{n} \left[ (\sigma_{si} - \sigma_c)A_{si} \left( \frac{d}{2} - d_{si} \right) \right]$$  \hspace{1cm} (5-2)

$$\phi = \frac{\varepsilon_{c,max}}{x}$$  \hspace{1cm} (5-3)

![Figure 5.1 Stress and strain over column depth](image-url)
5.2.3 Constitutive relationships of confined concrete

The tensile strength of concrete and the tension stiffening of reinforced concrete are relatively small and greatly degraded under seismic loading. Thus, these two effects of concrete in tension are considered negligible in this study. The compressive stress-strain relationships adopted in the computation program SAC for the concrete transversely confined by steel or FRP wrapping are introduced respectively in the following sections.
5.2.3.1 Constitutive relationship of steel-confined concrete

The secant modulus of elasticity $E_c$ and the strain $\varepsilon'_c$ corresponding to the compressive strength $f'_c$ (MPa) of unconfined concrete in the program SAC are determined by the following equations, which were proposed by Carrasquillo et al. (1981) and Popovics (1970), respectively:

$$E_c = 3320\sqrt{f'_c} + 6900 \quad \text{(MPa)} \quad (5-4)$$

$$\varepsilon'_c = \frac{f'_c}{E_c \left( \frac{c}{c-1} \right)} \quad (5-5)$$

where the parameter $c = 0.8 + \frac{f'_c}{17}$. In steel-confined concrete, the transverse confining stress remains basically unchanged or only increases insignificantly with the expansion of concrete after the yield of transverse steel reinforcement. Therefore, the confinement is deemed constant approximately. Based on extensive study, the stress-strain model proposed by Attard and Setunge (1996), which has been introduced in Section 2.2.1, is adopted in the program SAC for the concrete transversely confined by constant pressure. The ultimate strain $\varepsilon_{cu}$ of steel-confined concrete is defined as the strain at the first fracture of transverse reinforcement, which can be predicted by the following expression proposed by Priestley et al. (1996).

$$\varepsilon_{cu} = 0.004 + \frac{1.4 \rho_s \cdot f_{yh} \cdot \varepsilon_{su}}{f'_c} \quad (5-6)$$

where,

- $\rho_s$ = volumetric ratio of confining steel to concrete core in circular columns; For rectangular columns, $\rho_s = \rho_x + \rho_y$, in which $\rho_x$ and $\rho_y$ are the area ratio of confining steels to concrete core in x and y directions, respectively;
- $f_{yh}$ = yield strength of transverse confining steel;
- $\varepsilon_{su}$ = ultimate strain of transverse confining steel;
- $f'_c$ = compressive strength of steel-confined concrete.

The stress-strain model proposed by Attard and Setunge (1996) is also used for the unconfined concrete cover by setting the confining pressure $f_l$ as zero. The ultimate strain $\varepsilon_{cu}$ of the cover concrete at complete spalling is approximately taken as 0.005 based on test observations.
Due to the discontinuity of transverse confining steel, the area of effectively confined concrete is less than the total concrete core. The confinement efficiency model introduced in Section 2.2.1 is used in the program SAC for steel-confined columns.

![Figure 5.3 Stress-strain relationship of steel-confined concrete](image)

To illustrate the above introduced constitutive model of steel-confined concrete, the analytical stress-strain relationships of five 356 mm diameter circular columns are presented in Figure 5.3 as examples. These columns have no longitudinal reinforcement and are subjected to concentric loads. The compressive strength of unconfined concrete is 40 MPa. One of the columns is a control specimen without any confinement, while the other four columns have circular spirals of US #3 steel ($f_y = 490$ MPa) at spacing $s = 75$ mm, 100 mm, 150 mm and 300 mm, respectively, with clear concrete cover of 20 mm measured to the outer surface of spirals. These examples indicate that the strength and deformability of concrete is significantly improved by transverse confinement. This effect increases gradually with the decrease of spiral spacing. This model accurately captures the constitutive relationship of concrete of a wide range of strength and subjected to constant confinement, as verified by Cui (2009).

### 5.2.3.2 Constitutive relationship of FRP-confined concrete

Many constitutive models have been proposed to predict the behaviour of the FRP-confined concrete. The Cui and Sheikh model with an incremental procedure (Cui and Sheikh, 2010b) is used in the program SAC to get the analytical constitutive relationship for the FRP-confined
concrete. The selection of this model is mainly based on two considerations: one is its better accuracy verified by an extensive test data and the other is its applicability to a wide range of concrete strength (from 20 MPa to 120 MPa) and various types of FRP materials. The key points of the Cui and Sheikh model were introduced in Section 2.2.2.

For FRP-confined columns with noncircular cross section, the geometric efficiency should be considered. The method recommended by ACI 440.2R-08 is used in the program SAC to estimate the efficiency of FRP-confinement in columns with rectangular cross sections, which was introduced in Section 2.2.2.

![Stress-strain relationship of FRP-confined concrete](image)

**Figure 5.4 Stress-strain relationship of FRP-confined concrete**

To illustrate the constitutive relationship of FRP-confined concrete determined using the Cui and Sheikh model, analytical stress-strain curves for five 356 mm diameter circular columns are presented in Figure 5.4 as examples. These columns have no steel reinforcement and are subjected to concentric loads. The compressive strength of unconfined concrete is 40 MPa. One column is a control specimen without any confinement, while the other four columns are transversely confined by one or two layers of GFRP or CFRP jackets, respectively. The mechanical properties of the CFRP and GFRP jackets were presented in Table 3.2 of Chapter 3. The presented stress-strain curves are very close to the experimental observation reported by Cui (2009). Comparison with the constitutive relationships of steel-confined concrete in Figure 5.3 shows that the response of concrete with sufficient FRP-confinement has very different shape and almost no post-peak descending branch, except for the concrete confined by only one layer
of GFRP jacket. This is one of the reasons that lead to the different seismic behaviour of FRP-confined columns compared to the steel-confined counterparts.

5.2.4 Constitutive relationships of steel

5.2.4.1 Longitudinal reinforcement in tension

Normally the deformed reinforcing steel with flat yield plateau is used as the longitudinal reinforcement in concrete columns. The constitutive relationship measured in steel coupon tension tests can be used only for the longitudinal reinforcement in tension theoretically, which is modeled in this program to be linearly elastic-perfectly plastic while taking into account the strain-hardening as a parabolic curve by the following expressions:

\[
\sigma_s = E_s \varepsilon_s \quad \text{(for } 0 \leq \varepsilon_s \leq \varepsilon_y) \quad (5-7)
\]

\[
\sigma_s = f_y \quad \text{(for } \varepsilon_y < \varepsilon_s \leq \varepsilon_{sh}) \quad (5-8)
\]

\[
\sigma_s = f_y + (f_u - f_y) \cdot \left[ 2 \left( \frac{\varepsilon_s - \varepsilon_{sh}}{\varepsilon_{su} - \varepsilon_{sh}} \right) \left( \frac{\varepsilon_{su} - \varepsilon_{sh}}{\varepsilon_{su} - \varepsilon_{sh}} \right)^2 \right] \quad \text{(for } \varepsilon_{sh} < \varepsilon_s \leq \varepsilon_{su}) \quad (5-9)
\]

where

- \( \sigma_s \) = stress of steel corresponding to strain \( \varepsilon_s \) (MPa);
- \( f_y \) = yield strength of steel (MPa);
- \( f_u \) = ultimate strength of steel (MPa);
- \( \varepsilon_{sh} \) = strain at the beginning of strain-hardening;
- \( \varepsilon_{su} \) = ultimate strain corresponding to \( f_u \);
- \( \varepsilon_y \) = yield strain of steel;
- \( E_s \) = Young’s modulus of steel (MPa).

5.2.4.2 Longitudinal reinforcement in compression

The constitutive relationship of the longitudinal steel reinforcement in compression is deemed approximately to be the same as that in tension before yielding. However, the phenomenon of post-yield buckling of the steel bars in compression has to be considered to avoid overestimating the strength and deformability of columns.
The observation in the experiments introduced in Chapter 4, as well as in previous column tests (Sheikh and Khoury 1993; Sheikh et al. 1994; Bayrak and Sheikh 1998), indicated that the longitudinal reinforcement in compression displayed post-yield buckling in almost all specimens before failure. Bayrak and Sheikh (2001) suggested that the stress-strain relationship obtained from coupon tension tests could not be adopted directly for the steel reinforcement in compression.

The model proposed by Dhakall and Maekawa (2002) for the post-yield buckling of the steel in compression is used in the program SAC. In this model, the values of stress and strain corresponding to the stress-strain relationship of the steel in tension are referred to as point-wise compressive stress and strain, while the stress and strain of the steel in post-yield buckling are defined as the average compressive stress and strain. The relationship between the average compressive stress $\sigma_s$ and strain $\varepsilon_s$ is illustrated in Figure 5.5 and established by the following expressions:

$$\frac{\sigma_s}{\sigma_t} = 1 - \left(1 - \frac{\sigma_t^*}{\sigma_t}\right) \cdot \left(\frac{\varepsilon_s - \varepsilon_y}{\varepsilon_t^* - \varepsilon_y}\right) \quad \text{(for } \varepsilon_y \leq \varepsilon_s \leq \varepsilon_t^*) \quad (5-10)$$

$$\sigma_s = \sigma_t^* - 0.02E_s \cdot \left(\varepsilon_s - \varepsilon_t^*\right) \geq 0.2f_y \quad \text{(for } \varepsilon_t^* < \varepsilon_s \leq \varepsilon_{fu}) \quad (5-11)$$

where,

- $\varepsilon_t^*$ = strain at intermediate point;
- $\varepsilon_y$ = yield strain of steel;
- $\sigma_t$ = point-wise stress corresponding to current strain $\varepsilon_s$;
- $\sigma_t^*$ = point-wise stress corresponding to strain $\varepsilon_t^*$ at intermediate point;
- $\sigma_t^*$ = average stress corresponding to strain $\varepsilon_t^*$ at intermediate point;
- $E_s$ = Young’s modulus of steel (MPa);

The key point in this model is the definition of the intermediate point $(\varepsilon_t^*, \sigma_t^*)$ in the average stress-strain curve. The coordinates of this point are determined as:

$$\frac{\varepsilon_t^*}{\varepsilon_y} = 55 - 2.3, \sqrt{\frac{f_y}{100}} \cdot \left(\frac{s}{d_b}\right) \geq 7 \quad (5-12)$$
\[
\frac{\sigma^*}{\sigma_i} = \alpha \cdot \left(1.1 - 0.016 \sqrt{\frac{f_y}{100}} \left(\frac{s}{d_b}\right)\right) \geq 0.2 \cdot \left(\frac{f_y}{\sigma_i}\right)
\]

(5-13)

where,

- \( f_y \) = yield strength of longitudinal reinforcement (MPa);
- \( s \) = spacing of transverse reinforcement (mm);
- \( d_b \) = diameter of longitudinal reinforcement (mm);
- \( \alpha \) = factor to account the effect of different strain hardening character.

It was suggested that \( \alpha = 0.75 \) for the linearly elastic-perfectly plastic steel, while \( \alpha = 1.0 \) for the steel with linear hardening (Dhakall and Maekawa, 2002). In this program, \( \alpha \) is taken as 0.9 because the steel is supposed to have a parabolic strain-hardening.

![Figure 5.5 Constitutive relationship of steel reinforcement in compression](image)

An example is presented in Figure 5.6 to illustrate this model. The longitudinal reinforcement is a Grade 400 25M rebar which was used in the experiments introduced in Chapter 3. The post-yielding stress of the 25M rebar in compression is much less than its original point-wise stress. When the unsupported length of the 25M rebar is 300 mm \((s/d_b = 12)\), the post-yielding stress descended rapidly due to the compressive buckling, which is consistent with the observations reported by Bayrak and Sheikh (2001).
5.2.4.3 Transverse steel reinforcement

The constitutive relationship of the transverse steel reinforcement is taken as linearly elastic-perfectly plastic. That means the confinement is established based on the yield strength of transverse steel reinforcement and keeps constant throughout testing. This assumption is conservative for the concrete columns with closely spaced normal-strength transverse steel reinforcement. In the tests introduced in prior chapters, all the columns transversely confined by closely spaced spirals failed due to the rupture of spirals within the plastic hinge regions. Only in the final stages of testing, transverse steel in the columns goes into strain-hardening. The steel stress was thus assumed to be equal to the yield strength during the entire test, conservatively.

If high-strength steel was used to provide confinement in columns, the reliable stress in the transverse steel could only be up to 800 MPa (Bayrak, 1998). This limit is also adopted in the program SAC. It is noted that the specified upper limit of the steel strength in different concrete design codes still varies considerably. In seismic design provisions of ACI 318-11, the yield strength of transverse steel is limited to be no more than 690 MPa (100 ksi) in the design of confining reinforcement; while this limit is specified to be 500 MPa and 800 MPa in the seismic design provisions of CAN/CSA-23.3-04 and NZS 3101:2006, respectively.
5.3 Lateral shear vs. deflection analysis

5.3.1 Introduction

The structural model in the program SAC is a cantilever concrete column fixed on a rigid footing, subjected to a constant axial load and a monotonically increased lateral shear at the tip.

The total lateral tip deflection, $\Delta$, of the cantilever column consists of three components: (1) the flexural deflection, $\Delta_{\text{flexure}}$, due to the flexural deformation of column; (2) the slip deflection, $\Delta_{\text{slip}}$, owing to the fix end rotation of the column caused by tension slip of the longitudinal reinforcement out of the rigid footing; (3) the shear deflection, $\Delta_{\text{shear}}$, because of the shear deformation of the column, shown as

$$\Delta = \Delta_{\text{flexure}} + \Delta_{\text{slip}} + \Delta_{\text{shear}}$$  \hspace{1cm} (5-14)

The $P-\Delta$ effect has been taken into account when determining the moment along the column length. The moment at each section is composed of the contribution from lateral shear at the column tip and that from the $P-\Delta$ effect. A significant part of the flexural strength at the critical section may be consumed by the $P-\Delta$ effect when a slender column is subjected to a high level of axial load. Because the $P-\Delta$ effect depends on the deflection shape of the column, an iterative procedure is necessary to obtain the converged deflection profile for each given curvature at the critical section.

5.3.2 Flexural deflection

For a given curvature distribution $\phi(x)$ along the column height $L$, the component of lateral tip deflection due to the flexural deformation is obtained by the integration as:

$$\Delta_{\text{flexure}} = \int_0^L \phi(x)x\,dx$$  \hspace{1cm} (5-15)

where $x = \text{distance of each section to the tip of column}$.

This method can directly be used to estimate the lateral flexural deflection, $\Delta_{\text{flexure}}$, at the column tip when the curvature at the critical section is within the ascending branch of the moment vs. curvature ($M-\phi$) envelope curve. However, an assumption about the development of plastic
hinge should be integrated into the algorithm when the critical curvature goes into the post-peak branch of the $M-\phi$ curve. This issue will be addressed later.

5.3.3 Slip deflection

Sufficient anchorage length is assumed for longitudinal reinforcement in this analysis procedure. As reported in Chapter 4, noticeable cracking and some bar slip were observed on the tension side of the columns right at the column-footing interface during testing, even though the longitudinal bars were extended to close to the end of stubs with enough anchorage length. This phenomenon indicates that the yield penetration and slip of the tensile longitudinal steel out of the footing cause an extra elongation of the tensile steel at the critical section. For simplification, it is assumed that the cracking section at the column-stub interface remains plane and the column rotates around the neutral axis for an angle $\theta_{\text{slip}}$ in the column-footing interface. The resulted rigid rotation $\theta_{\text{slip}}$ of the whole column leads to an extra tip deflection $\Delta_{\text{slip}}$, which may contribute a significant part to the total tip deflection $\Delta$. In a $fib$ state-of-the-art report, the results from a test series showed that the lateral deflection of columns was increased by 59% on average due to the pull-out of longitudinal reinforcement from the concrete footing ($fib$ Task Group 7.2, 2003).

The slip deflection $\Delta_{\text{slip}}$ at the tip of column is estimated as follows:

$$\theta_{\text{slip}} = \frac{\delta_{st}}{d-c} \quad (5-16)$$

$$\Delta_{\text{slip}} = \theta_{\text{slip}} \cdot L \quad (5-17)$$

where

- $d$ = distance from extreme compression fiber of concrete to centroid of extreme layer of longitudinal tensile steel;
- $c$ = distance from extreme compression fiber to neutral axis at the fixed end of column;
- $L$ = height of cantilever columns;
- $\delta_{st}$ = slip extension of extreme tensile longitudinal steel of column out of the fixed footing.

The slip extension of the embedded longitudinal reinforcement, $\delta_{st}$, is the result of integrating the strain along the whole anchorage length when ignoring the tiny deformation of the concrete footing. For the steel used in this analysis procedure, $\delta_{st}$ may consist of two components: the slip
coming from the elastic region of steel, $\delta_{se}$, and that from the plastic region of steel, $\delta_{sp}$, which are estimated using the following method proposed by Alsiwat and Saatioglu (1992).

$$\delta_{se} = \delta_{ste} + \delta_{sp}$$ \hspace{1cm} (5-18)

Before the yield of the embedded tensile steel, there is no plastic region and the extension $\delta_{st}$ of the steel comes solely from the elastic region as:

$$\delta_{ste} = 0.5 \varepsilon_s L_e$$ \hspace{1cm} (for $\varepsilon_s \leq \varepsilon_y$) \hspace{1cm} (5-19)

where

- $\varepsilon_s$ = strain of longitudinal reinforcement at the critical section;
- $\sigma_s$ = stress of longitudinal reinforcement at the critical section (MPa);
- $L_e = \frac{\sigma_s d_b}{4 u_e}$, length of the elastic extension region when $\sigma_s \leq f_y$, (mm);
- $u_e = \frac{f_y d_b}{4 L_d}$, average friction stress in the elastic extension region (MPa);
- $L_d = \frac{440 A_b}{400 f_c} \geq 300$ mm, maximum length of the elastic extension region (mm);
\( A_b = \) cross-sectional area of a longitudinal steel bar (mm\(^2\));
\( d_b = \) diameter of longitudinal steel reinforcement (mm);
\( f'_c = \) compressive strength of concrete in footing (MPa);
\( f_y = \) yield strength of longitudinal reinforcement (MPa);
\( \varepsilon_y = \) yield strain of longitudinal reinforcement;
\( K = \) factor, taken as 3\( d_b \) (mm).

After the yield of the embedded tensile steel, its extension \( \delta_{ste} \) of the elastic region is estimated by taking \( \sigma_s = f_y \) in the above procedure. Furthermore, after the occurrence of strain-hardening of the embedded tensile steel, the extension \( \delta_{stp} \) from the plastic region should also be added into the total slip extension \( \delta_s \).

The amount of \( \delta_{stp} \) is estimated approximately as:

\[
\delta_{stp} = 0.5 (\varepsilon_s + \varepsilon_{sh}) L_{sh} \quad \text{for} \quad \varepsilon_s > \varepsilon_{sh}
\]

where
\( \varepsilon_{sh} = \) strain at the beginning of strain-hardening of longitudinal reinforcement;
\( L_{sh} = \frac{(\sigma_s - f_y) d_b}{4u_f} \), length of plastic extension region when \( \sigma_s > f_y \), (mm);
\( u_f = \left( 5.5 - 0.07 \frac{S_L}{H_L} \right) \frac{f'_c}{27.6} \), average friction stress in plastic extension region, (mm);
\( S_L = \) clear spacing of ribs on longitudinal steel bars (mm);
\( H_L = \) height of ribs on longitudinal steel bars, taken as 0.125\( S_L \) in this research, (mm).

5.3.4 Shear deflection

The shear deformation of the concrete column can be estimated accurately but also laboriously by the Modified Compression Field Theory (MCFT) developed by Vecchio and Collins (1986). In this research, the seismic behaviour of columns is mainly controlled by the flexural effect. The component of the tip deflection due to the shear deformation of columns is much less than the other two deflection components. Thus, a simplified shear deformation model is adopted in the analysis procedure as follows.

Before cracking, concrete is assumed to be isotropic linearly elastic and the tip deflection due to
the shear deformation, $\Delta_{\text{shear}}$, is estimated approximately using the elastic theory as:

$$\Delta_{\text{shear}} = \frac{VL}{AG} \tag{5-21}$$

where

- $V$ = lateral shear acting at the column tip (N);
- $L$ = height of cantilever column (mm);
- $A$ = cross-sectional area of column (mm$^2$);
- $G = \frac{E_c}{2(1+\nu)}$, shear modulus of concrete (MPa);
- $\nu$ = Poisson’s ratio, taken as 0.20 for concrete before cracking.

After cracking, both the effective shear modulus and effective shear area of the concrete decrease dramatically. The deflection $\Delta_{\text{shear}}$ is estimated by using a simplified method proposed by Lehman and Moehle (2000) as:

$$\Delta_{\text{shear}} = \int_0^L \left( \frac{V(x)}{A_{\text{eff}}(x) \cdot G_{\text{eff}}(x)} \right) dx = V \cdot \int_0^L \frac{dx}{A_{\text{eff}}(x) \cdot G_{\text{eff}}(x)} \tag{5-22}$$

where

- $A_{\text{eff}}(x)$ = effective shear area of column, taken as area of the confined concrete core which is subjected to compression (mm$^2$);
- $E_{c,\text{eff}}(x)$ = effective secant modulus of elasticity of concrete, assessed from the compressive stress and strain at extreme compression fiber of concrete core (MPa);
- $G_{\text{eff}}(x) = \frac{E_{c,\text{eff}}(x)}{2(1+\nu)}$, effective shear modulus of core concrete (MPa);
- $\nu$ = Poisson’s ratio of core concrete, taken as 0.30 after concrete cracking.

In the above computation, the concrete core is measured to the centreline of the peripheral transverse steel for columns solely confined by transverse steel rebars; while the whole cross section is taken as the confined concrete core in FRP-confined columns.

### 5.3.5 Assumption of plastic hinge

The estimation of the flexural deflection, $\Delta_{\text{flexure}}$, cannot be carried out directly using the
straightforward integration, as stated in Section 5.3.2, after the curvature at the critical section goes into the post-peak branch of $M-\phi$ curve, due to the complicated distribution of curvature within the most damaged region. The following assumption about the development of plastic hinge has to be implemented into the algorithm to overcome this barrier.

5.3.5.1 Development of plastic hinge

The plastic hinge region is defined at the location adjacent to the critical section of column. Within the plastic hinge region, the curvature is assumed to be uniform and equals the curvature at the critical section, while the curvature at section outside plastic hinge region is determined by the corresponding moment according to the ascending branch of $M-\phi$ curve.

To obtain the whole curve of $V-\Delta$ relationship of the column, the incremental procedure is controlled by the curvature at the critical section, which increases from zero to the maximum possible value or until the lateral force $V$ decreases to lower than 50% of the nominal lateral force capacity $V_n$ along the descending branch of $V-\Delta$ curve, whichever occurs first. In this procedure, the development of the plastic hinge length is based on the following points:

1. There is no plastic hinge when the curvature at the maximum moment section of columns is less than the yield curvature $\phi_y$.

2. The plastic hinge length develops from zero to its full amount $l_p$ when the curvature at the maximum moment section increases from the yield curvature $\phi_y$ to the curvature $\phi_c$ corresponding to the peak moment $M_{max}$. The estimation of $l_p$ is discussed in the next section.

3. After the curvature at the maximum moment section exceeds $\phi_c$, the length of plastic hinge will keep constant as $l_p$ until the termination of lateral shear vs. deflection analysis.

5.3.5.2 Plastic hinge length

It should be clarified that there are two different concepts of the plastic hinge length $l_p$ in the published literature. One is the length of the equivalent plastic hinge with which the curvature ductility factor $\mu_\phi$ of the column is directly related to the displacement ductility factor $\mu_\Delta$, such as
in the following expressions proposed by Paulay and Priestley (1992):

\[ \mu_L = 1 + 3(\mu_p - 1) \frac{l_p^L}{L} \left( 1 - 0.5 \frac{l_p^L}{L} \right) \]  
\[ l_p^L = 0.08 L + 0.022 d_b f_y \]

where:

- \( d_b = \) diameter of longitudinal reinforcement (mm);
- \( f_y = \) yield strength of longitudinal reinforcement (MPa);
- \( L = \) height of cantilever columns (mm).

This equivalent hinge length \( l_p \) was derived from experimental observations and the allowance was made for the tip deflection components due to the slip of tension steel, \( \Delta_{slip} \), and the shear deformation, \( \Delta_{shear} \), of the column.

Another concept of the plastic hinge length is aimed at the computation of the column tip deflection solely due to the flexural deformation, \( \Delta_{flexure} \), as used in the program SAC. Theoretically, the length of the plastic hinge should reflect the length of the observed most damaged region of column due to plastic deformation. It has been found that the length of the most damaged region varied from \( 0.4D \) to \( 1.2D \) in tests, where \( D = \) diameter of circular columns or depth of square columns. Bae and Bayrak (2008) reported that the length of the plastic hinge increased with the enhancement of transverse confinement, the shear span-depth ratio and the axial load level of the column.

The output of the program SAC showed that the simple assumption of \( l_p = 1.0D \) led to reasonably good estimation of the tip deflection for well-confined columns. Different lengths of the plastic hinge \( l_p \) were tried during the calibration process. It was found that for the columns analyzed in this study, the predicted \( V-\Delta \) curves had only minor difference when \( l_p \) varied from \( 0.6D \) to \( 1.0D \). In most cases, the predicted deflection curves were more realistic with \( l_p = 1.0D \), as shown by the examples in Figure 5.8. Thus, the length of the plastic hinge \( l_p \) is taken as \( 1.0D \) in the program SAC.
Figure 5.8 Comparison of different assumptions of plastic hinge length
5.4 Verification of computation program

5.4.1 Introduction

A large number of near full-scale concrete columns with square and circular cross sections were tested under simulated seismic loading at the University of Toronto in the past two decades. These test results, together with the collected test data from other published literature as listed in Table 2.26 and Table 2.27, are used to verify the program SAC.

Based on the experimental results of normal-strength and high-strength concrete reported by Karsan and Hirsa (1969) and Bing et al (1994), it is known that the envelope curve of stress-strain hysteresis loops of concrete under cyclic axial loading can be represented by the stress-strain curve measured in the corresponding monotonic loading test approximately. Thus, it is concluded that the envelope curves of the hysteresis loops of moment vs. curvature ($M$-$\phi$) and shear vs. tip deflection ($V$-$\Delta$) relationships of the concrete columns under lateral cyclic loads are practically coincident with the responses of columns under corresponding monotonic lateral loads. The following verification is based on the comparison between the results of the monotonic pushover analysis by the program SAC and the envelope curves of the experimental responses.

5.4.2 Steel-confined columns

5.4.2.1 Steel-confined circular columns

The moment vs. curvature ($M$-$\phi$) and shear vs. tip deflection ($V$-$\Delta$) responses are shown in Figure 5.9 and Figure 5.10 for four normal strength concrete (NSC) columns, P27-NF-1, P27-NF-2, P40-NF-6 and P56-NF-11 tested in this study and six high strength concrete (HSC) columns C100S100N15, C100SH100N15, C100S70N25, C100SH70N25, C100S37N40 and C100SH37N40 tested by Paultre et al. (2009) with concrete strength of 100 MPa. The analytical curves using SAC are also shown in the figures. A comparison of responses shows a reasonable agreement between the analytical curves and the envelope curves of experimental results of columns with a wide range of concrete strength.
5.4.2.2 Steel-confined square columns

A large database is available on steel-confined square columns which can be used to verify the program SAC. The results of six NSC columns AS-3, FS-9, ES-13, AS-17, AS-18, AS-19 tested by Sheikh and Khoury (1993) and three HSC columns AS-3H, AS-18H, AS-20H tested by Sheikh et al. (1994) are presented as examples in this section.

In Figure 5.11, the predicted $M$-$\phi$ curves are found to be in good agreement with the envelope curves of the measured test results for all these specimens. The flexural strength, elastic and plastic deformation characteristics of the columns are predicted with good accuracy.

Similarly, as shown in Figure 5.12, the measured $V$-$\Delta$ envelope curves can be captured by the program SAC very well. The maximum lateral shear forces and the ascending and descending branches of the $V$-$\Delta$ envelope curves are predicted with reasonable accuracy.

5.4.3 FRP-confined columns

5.4.3.1 FRP-confined circular columns

The comparisons between the experimental and analytical results of ten specimens are presented in Figure 5.13 and Figure 5.14 to show the verification of the program SAC for the FRP-confined circular columns. Specimens P27-1CF-3, P27-2GF-4, P40-1CF-8, P40-1GF-9, P56-3GF-14 and P27-4GF-15 are part of the current study while specimens ST-2NT, ST-3NT, ST-4NT and ST-5NT were tested by Sheikh and Yau (2002). These columns were transversely confined by CFRP or GFRP wrapping in the potential plastic hinge regions.

The comparison indicates the excellent simulation of experimental responses by the program SAC in all these cases. The least favourable comparison is the $V$-$\Delta$ curve of column ST-3NT, which was confined by one layer of CFRP jacket and subjected to a axial loading level of $k_p = P/P_o = 0.50$. However, if compared with the other similar specimens, such as ST-2NT, P56-2CF-13 and P27-3GF-14, it is found that the measured initial stiffness of ST-3NT was much lower than all the other comparable specimens. Possible experimental error in the measurement of the $V$-$\Delta$ relationship of ST-3NT may be the reason behind this discrepancy. It should be noted that the moment vs. curvature response of this specimen has been predicted quite well.
5.4.3.2 FRP-confined square columns

The available database on experimental results is still quite limited for the large-scale FRP-confined square columns tested under simulated seismic loading. Here, the verification of the program SAC is conducted by using five GFRP-confined square columns and five CFRP-confined square columns.

The five GFRP-confined square columns ASG-2NSS, ASG-3NSS, ASG-4NSS, ASG-5NSS and ASG-6NSS were tested by Momen and Sheikh (2005), while the experimental results of the five CFRP-confined square columns ASC-2NS, ASC-3NS, ASC-4NS, ASC-5NS and ASC-6NS were reported by Iacobucci et al. (2003). As shown in Figure 5.15 and Figure 5.16, all the measured \( M-\phi \) and \( V-\Delta \) envelope curves are predicted well. Especially, the accuracy of the predicted \( V-\Delta \) envelope curves is better than that of the \( M-\phi \) curves.
Figure 5.9 Experimental and analytical $M$-$\phi$ curves of steel-confined circular columns
Figure 5.10 Experimental and analytical $V$-$\Delta$ curves of steel-confined circular columns
Figure 5.11 Experimental and analytical $M-\phi$ curves of steel-confined square columns
Figure 5.12 Experimental and analytical $V$-$\Delta$ curves of steel-confined square columns
Figure 5.13 Experimental and analytical $M$-$\phi$ curves of FRP-confined circular columns
Figure 5.14 Experimental and analytical $V$-$\Delta$ curves of FRP-confined circular columns
Figure 5.15 Experimental and analytical $M$-$\phi$ curves of FRP-confined square columns
Figure 5.16 Experimental and analytical $V$-$\Delta$ curves of FRP-confined square columns
5.4.4 Strength and deformation – Analytical vs. Experimental

The computation program SAC has initially been verified by comparisons between the analytical results of the pushover analysis and the measured experimental $M-\phi$ and $V-\Delta$ envelope curves. To describe the seismic behaviour of concrete columns quantitatively, the commonly used parameters include the flexural strength $M_{\text{max}}$, maximum lateral shear $V_{\text{max}}$, curvature ductility factor $\mu_{\phi}$, displacement ductility factor $\mu_{\Delta}$, and lateral drift ratio $\delta$. The accuracy of predictions for these parameters is discussed in the following sections, by using the test results of all the experiments presented in Table 2.26 and Table 2.27.

5.4.4.1 Strength of columns

Among all the previously mentioned parameters, the flexural strength $M_{\text{max}}$ and the corresponding maximum lateral shear $V_{\text{max}}$ of columns can be predicted with very good accuracy by the program SAC. This is shown by the comparisons in Figure 5.17 and Figure 5.18.

For steel-confined columns, the ratios of the analytical prediction to the measured flexural strength $M_{\text{max}}$ have an average of 1.069 with a standard deviation of 0.069, while the ratios of the analytical prediction to the measured lateral shear $V_{\text{max}}$ have an average of 0.992, with a standard deviation of 0.072. For FRP-confined columns, the ratios of the analytical prediction to the
measured flexural strength $M_{\max}$ have an average of 1.074 with a standard deviation of 0.094, while the ratios of the analytical prediction to the measured maximum shear $V_{\max}$ have an average of 0.976, with a standard deviation of 0.089. The predictions of the strength $M_{\max}$ and $V_{\max}$ of concrete columns are in good agreement with the measured test results.

![Graphs showing the comparison between predicted and measured flexural strength and shear demand.](image)

(a) Flexural strength  
(b) Shear demand

Figure 5.18 Predicted and measured strength of FRP-confined columns

### 5.4.4.2 Curvature ductility factor

The curvature ductility factor $\mu_\phi$ of the column, defined as the ratio of the ultimate curvature $\phi_u$ to the yield curvature $\phi_y$, has been recognized as the most fundamental ductility parameter and used as the criterion to develop seismic design provisions for the concrete design codes such as CAN/CSA-A23.3-04, CAN/CSA-S806-12 and NZS 3101:2006.

In Figure 5.19, the predicted curvature ductility factors $\mu_\phi$ by the program SAC are compared with the corresponding experimental results. The predictions of all the columns show reasonable accuracy. For steel-confined columns, the ratios of the analytical prediction to the measured $\mu_\phi$ results have an average of 1.017, with a standard deviation of 0.276. While for FRP-confined columns, the ratios of the analytical prediction to the measured $\mu_\phi$ results have an average of 1.037, with a standard deviation of 0.289.
It should be noted that the accurate measurement of curvature ductility in the plastic hinge region is known to be difficult at large deformations, mainly due to the uneven development of the curvature in this region. Furthermore, when columns approach failure, the granulation of concrete makes it even harder to keep the measuring instruments in place firmly throughout testing and to measure the plastic deformation completely. This always leads to the scattered measurement of the curvature ductility factor $\mu_\Phi$ and may explain why the experimental results of $\mu_\Phi$ were not reported in a lot of scientific literature.

**5.4.4.3 Displacement ductility factor**

The predicted displacement ductility factors $\mu_\Delta$ by the program SAC are compared with the experimental results in Figure 5.20. The predictions of all the specimens are reasonably accurate, while the results of square columns are less scattered than those of circular columns. For steel-confined columns, the ratios of the analytical prediction to the experimental values of $\mu_\Delta$ have a mean of 1.032, with a standard deviation of 0.221. While, for FRP-confined columns, the ratios of the analytical prediction to the experimental values of $\mu_\Delta$ have a mean of 1.041, with a standard deviation of 0.253.
5.4.4.4 Lateral drift ratio

Lateral drift ratio $\delta$ is used by some researchers and design codes, such as CAN/CSA-S806-12, as a seismic design criterion of the FRP-confined concrete columns.

Figure 5.20 Predicted and measured displacement ductility factors

(a) Steel-confined columns  
(b) FRP-confined columns

Figure 5.21 Predicted and measured lateral drift ratios

(a) Steel-confined columns  
(b) FRP-confined columns
In Figure 5.21, the predictions of the lateral drift ratio $\delta$ by the program SAC are compared with the measured results. For steel-confined columns, the ratios of the analytical prediction to the measured $\delta$ results have a mean of 0.997, with a standard deviation of 0.255. While for FRP-confined columns, the ratios of the analytical prediction to the measured $\delta$ results have a mean of 0.956, with a standard deviation of 0.133. Considering the difficulty in the experimental measurement, the analytical results show reasonable accuracy.

5.4.5 Summary

The program SAC is capable of capturing the main characteristics of the seismic behaviour of confined concrete columns. Among the commonly used parameters, the strength of columns ($M_{\text{max}}$, $V_{\text{max}}$) can be predicted with high accuracy, while the predictions for curvature ductility factor, displacement ductility factor and lateral drift ratio of the column are reasonably accurate. The computational results may be deemed as the close representation of experimental results. Therefore, this computation program is used in the next chapter to conduct a numerical study on the concrete columns under seismic loading to overcome the limitation of the available experimental results of large-scale columns.
Chapter 6
Parametric Study of Column Behaviour

6.1 Introduction

A numerical parametric study is conducted in this chapter to investigate the seismic behaviour of confined concrete columns by using the program SAC. The columns have square or circular cross sections and are transversely confined by steel reinforcement or the FRP wrapping. Various factors such as concrete strength, type and amount of transverse reinforcement, amount of longitudinal reinforcement, and geometric properties of columns, are investigated for their effects on column behaviour.

Based on extensive computational results, the developing trends of the strength enhancement and different ductility factors of transversely confined concrete columns, which include the curvature ductility factor, displacement ductility factor and lateral drift ratio, are studied individually in following sections.

6.2 Parameter combination

6.2.1 Key variables

As shown by the experimental results in the previous chapters and published literature, the ductility capacity and strength of confined concrete columns are mainly influenced by the following factors: concrete strength $f'_c$, effective transverse confining pressure $f_t$, axial load level $k_p = P/P_o$, the area ratio of the concrete core to the gross cross section of the column, $A_c/A_g$, shear span-depth ratio $\lambda = L/D$, the mechanical ratio of longitudinal steel reinforcement $\rho_m = \rho_l \cdot (f_y/f'_c)$ and the ultimate strain $\varepsilon_{au}$ of transverse steel reinforcement, where: $A_g =$ gross cross-sectional area of the column; $A_c =$ cross-sectional area of concrete core, measured to the centerline of peripheral transverse steel reinforcement; $L =$ shear span of column; $D =$ diameter of circular columns or side dimension of square columns; $\rho_l =$ area ratio of the total longitudinal steel reinforcement to the gross cross section of the column; $m = f_y/f'_c =$ ratio of the yield strength of longitudinal steel reinforcement to the compressive strength of the unconfined concrete.

In the following numerical analysis, the influences of all these factors on the seismic behaviour
of confined concrete columns are studied using the program SAC described in Chapter 5.

### 6.2.2 Variables

The following ranges of variables are analyzed in the numerical study to cover the engineering practice as much as possible:

1. Transverse confinement level \( f_t / f'_c = 0.02, 0.05, 0.1 \) and 0.2;
2. Axial load level \( k_p = P/P_o = 0.1, 0.2, 0.3, 0.4, 0.5 \) and 0.6;
3. Compressive strength of unconfined concrete \( f'_c = 30, 45, \) and 60 MPa;
4. Shear span-depth ratio \( \lambda = L/D = 3.0, 4.0 \) and 5.0;
5. Mechanical ratio of the longitudinal reinforcement \( \rho_m = 0.1, 0.2, 0.3, \) and 0.4;
6. Four types of the column cross sections: square cross sections of 500×500 mm and 1000×1000 mm, and circular cross sections with 500 mm and 1000 mm diameter;
7. Three types of transverse confining material: steel rebars, CFRP jackets and GFRP jackets;
8. Clear concrete cover: \( cc = 30 \) and 40 mm for the steel-confined columns, while only \( cc = 40 \) mm for all the FRP-confined columns.

In summary, the numerical simulation is performed for the columns with a total of 13824 combinations of variables.

All the steel reinforcements in columns are assumed to be Grade 400 steel, with Young’s modulus \( E_s = 200 \) GPa, the yield strength \( f_y = 400 \) MPa and the ultimate strength \( f_u = 500 \) MPa. The strain-hardening begins at \( \varepsilon_{sh} = 0.01 \) and the ultimate strain is taken as \( \varepsilon_{su} = 0.10 \). These strain values are the lower-bound of the confining steel bars normally used in earthquake-resisting concrete structures (CAN/CSA-G30.18-09). Higher curvature ductility factor may be resulted from the higher ultimate strain of transverse confining steel. This issue will be discussed later in the study of curvature ductility factor of steel-confined columns.

In steel-confined columns, the spacing of transverse steel reinforcement is assumed to be no more than the smallest of \( 1/4 \) of the cross section size and 8 times the diameter of the longitudinal steel reinforcement and 24 times the diameter of the transverse steel reinforcement, which satisfies the relevant detailing requirements in CAN/CSA-A23.3-04 for steel-confined columns in moderately-ductile moment-resisting frames to avoid the occurrence of the severe premature buckling of longitudinal reinforcement.
The CFRP jackets have the ultimate tensile strength $f_{Fu} = 939$ MPa and Young’s modulus $E_F = 76433$ MPa, while for the GFRP jacket, the ultimate tensile strength $f_{Fu} = 518$ MPa and Young’s modulus $E_F = 25488$ MPa. These properties are the same as those of the FRP composites used in the experimental study in Chapter 3. The confinement contribution from any transverse steel is ignored in the FRP-confined columns assuming that only minimal confining steel was presented in these columns. All the corners of the FRP-confined square columns are assumed to be rounded to a radius of 35 mm before the FRP wrapping.

For columns subjected to axial load level $k_p = 0.1$, the ductility capacity was found to be mainly controlled by the ultimate tensile strain of the longitudinal steel rather than the confinement to concrete. Since this study specifically deals with the confinement of concrete, columns with $k_p = 0.1$ are excluded in the following numerical study.

It should be noted that the units of $f_c'$, $f_y$ and $f_{Fu}$ are in the unit of MPa throughout the following numerical analysis process in this chapter.

### 6.3 Curvature ductility factor

#### 6.3.1 Steel-confined columns

Numerical analysis is conducted in this section to evaluate the variations in curvature ductility factor $\mu_\phi$ of steel-confined circular and square columns. Analytical expressions are then developed based on the computational results.

#### 6.3.1.1 Parametric analysis

It has been well recognized that the curvature ductility factor $\mu_\phi$ of the concrete column is mainly influenced by three factors: the effective transverse confinement $f_l$, the concrete strength $f_c'$, and the axial load level $k_p$ applied on the column. It should be noted that the effective transverse confinement $f_l$ accounts for the cross-sectional shape of the column in addition to the amount and strength of transverse reinforcement. The computational results reveal that the effect of cross-sectional shape of the column on curvature ductility factor is correctly accounted for if the effective confining stress $f_l$ is estimated appropriately. The area ratio of concrete core to the gross
cross section of the column, $A_c/A_s$, appears to have no significant influence on the $\mu_\phi$ of steel-confined columns.

The capacity of $\mu_\phi$ improves with the increase of $f_l$ and with a reduction of $f_c'$ and $k_p$, as reported in the published literature (e.g. Sheikh and Khoury, 1993; Bayrak and Sheikh, 1998). This observation is also verified by most computational results from this study and can be expressed by a relationship between $\mu_\phi$ and $f_l/(f_c' \cdot k_p)$ for the steel-confined columns. As summarized in Figure 6.1, the regressed centric trend line of the data is

$$\mu_\phi = 90 \cdot \frac{f_l}{f_c' \cdot k_p}$$  \hspace{1cm} (6-1)

Figure 6.1 Relationship of $\mu_\phi$ vs. $f_l/(f_c' \cdot k_p)$ of steel-confined columns

However, the wide scatter of data in Figure 6.1 indicates that this expression cannot be used with sufficient confidence, even though a similar expression was also proposed by Paultre and Légeron (2008) and has been used to derive the design equations for CAN/CSA-A23.3-04. As an example, the curvature ductility factor $\mu_\phi$ varies between 9 and 27 for columns with confinement $f_l/(f_c' \cdot k_p) = 0.17$. Thus, further numerical study is necessary in order to predict the factor $\mu_\phi$ of columns more accurately.

Based on the comparison of computational results, it is found that the $\mu_\phi$ vs. $k_p$ relationship of steel-confined columns is significantly influenced by the mechanical ratio $\rho_m$ of longitudinal reinforcement. By using the computational results of a circular column of $f_c' = 30$ MPa, $D = 500$
mm, and cc = 30 mm as an example, following six points are made from Figure 6.2 in which the variations in the yield curvature $\phi_y$, the ultimate curvature $\phi_u$ and the curvature ductility factor $\mu_\phi$, as $k_p$ changes are shown for different values of $\rho_m$:

1) It is indicated by the first column of charts in Figure 6.2 that the relationship of $\phi_y$ vs. $k_p$ is significantly affected by $\rho_m$. The increase of $k_p$ from 0.2 to 0.6 only results in a gentle decrease of $\phi_y$ for columns of $\rho_m$ = 0.1. On the contrary, for columns of $\rho_m$ = 0.4 the $\phi_y$ decreases sharply with the increase of $k_p$.

2) The first column of charts in Figure 6.2 also shows that the influence of $\rho_m$ on $\phi_y$ is significantly affected by $k_p$. The $\phi_y$ increase from 6 to 9 with the increase of $\rho_m$ from 0.1 to 0.4 for columns under $k_p$ = 0.2. But for columns under $k_p$ = 0.6, the influence of $\rho_m$ on $\phi_y$ is minimal.

3) As shown in the second column of charts in Figure 6.2, the ultimate curvature $\phi_u$ always decreases with the increase of $k_p$ and this tendency is not significantly influenced by $\rho_m$, except one column with $\rho_m$ = 0.1, $k_p$ = 0.2 and $f_{li}/f'_c$ = 0.02 which fails due to the fracture of the longitudinal steel instead of the confined concrete and thus is beyond the current scope of research.

4) The charts in the second column of Figure 6.2 also show that the parameter $\rho_m$ does not affect the $\phi_u$ to the same degree as it does the $\phi_y$.

5) The relationship of $\mu_\phi$ vs. $k_p$ is presented in the third column of charts in Figure 6.2. It can be seen that the increase of $k_p$ from 0.2 to 0.6 results in a quick decrease of $\mu_\phi$ for columns with $\rho_m$ = 0.1, while $k_p$ has no significant influence on $\mu_\phi$ for columns with $\rho_m$ = 0.4. This phenomenon can be explained by the above-mentioned $\phi_y$ vs. $k_p$ relationship. For columns with $\rho_m$ = 0.4, if the value of $k_p$ increases from 0.2 to 0.6, the decrease of $\phi_u$ is accompanied by the decrease of $\phi_y$ in a similar degree, which results in the almost unchanged $\mu_\phi$.

6) The influence $\rho_m$ on $\mu_\phi$ is also revealed by the charts in the third column of Figure 6.2. For instance, for columns of $k_p$ = 0.2 and $f_{li}/f'_c$ = 0.05, the factor $\mu_\phi$ decreases from 24 to 13 if the parameter $\rho_m$ increases from 0.1 to 0.4. However, for columns of $k_p$ = 0.6 and $f_{li}/f'_c$ = 0.05, the factor $\mu_\phi$ decreases only from 12 to 10 when the parameter $\rho_m$ increases from 0.1 to 0.4. The
interactive influence of parameters $k_p$ and $\rho_m$ on the development of $\mu_\phi$ is indicated again by this phenomenon. It is noticed that above observations are contrary to the report by Watson et al. (1994) and the relevant provisions in the NZS 3101:2006 Code, in which it was stated that curvature ductility factor $\mu_\phi$ of columns always improves with the increase of $\rho_m$. Meanwhile, the influence of $\rho_m$ is ignored in the requirements for confinement design of columns in CAN/CSA-A23.3-04 and EN 1998-1:2004. The design provisions can be improved by considering the influence of $\rho_m$ appropriately.

Figure 6.2 Influence of $k_p$ and $\rho_m$ on $\mu_\phi$ of steel-confined columns
In summary, transverse confinement can improve the curvature ductility factor \( \mu_\Phi \) of steel-confined columns. But the improvement of \( \mu_\Phi \) is influenced by the parameters \( k_p \) and \( \rho m \) interactively. Considering all of these parameters is necessary to accurately predict the factor \( \mu_\Phi \) of steel-confined columns.

6.3.1.2 Proposed expressions

Based on the numerical analysis and comparison, it is found that the composite parameter \( \frac{f_i}{(f'_c)^{1.5}} \) can reflect the degree of influence of each variable of \( f_i \) and \( f'_c \) on the factor \( \mu_\Phi \) with the least scatter. The relationships of \( \mu_\Phi \) vs. \( \frac{f_i}{(f'_c)^{1.5}} \) are presented in Figure 6.3 by using the computational results of steel-confined columns with square and circular cross sections, under the axial load level \( k_p = 0.2 \) to 0.6, with concrete strength up to 60 MPa, and the ultimate strain of transverse steel \( \varepsilon_{su} = 0.1 \). For the purpose of engineering practice, the computational results of columns with impractically high level of \( \mu_\Phi \) and high confinement are excluded from the study, thus the effective data range is limited as \( \mu_\Phi < 40 \) and \( \frac{f_i}{(f'_c)^{1.5}} < 0.035 \).

In order to study the interactive influence of the axial load level \( k_p \) and the mechanical ratio of longitudinal reinforcement \( \rho m \), the relationship of \( \mu_\Phi \) vs. \( \frac{f_i}{(f'_c)^{1.5}} \) is categorized according to the parameters \( k_p \) and \( \rho m \). The data in each column of charts in Figure 6.3 represent the computational results of columns of the same \( \rho m \), while the date in each row of charts are the results of columns under the same \( k_p \). Obviously, \( k_p \) and \( \rho m \) are interactive in the same way as that was discussed in the previous example. The regressed centric trend line of data is labelled with its equation in each chart in Figure 6.3. Based on the equations of centric trend lines, following expression is proposed for the relationship between \( \mu_\Phi \) and \( \frac{f_i}{(f'_c)^{1.5}} \) of steel-confined columns:

\[
\mu_\Phi = k_1 \cdot \frac{f_i}{(f'_c)^{1.5}} + k_2
\]

(6-2a)
where:
\[
k_1 = 1163(k_p)^{k_3}
\]
\[
k_2 = 6 - 10k_p
\]
\[
k_3 = 1.83\rho lm - 0.54
\]

The above expression represents most of the trend lines in Figure 6.3 with good accuracy. Figure 6.4(a) compares \(\mu \phi\) values from Equation 6.2a for all the steel-confined columns with those obtained by the program SAC. SAC computes the moment-curvature response of the column section from which \(\mu \phi\) was calculated. The improved accuracy of Eq.(6-2a) is shown clearly by the convergence of data, comparing with the results in Figure 6.1.

Furthermore, it is found that the ultimate strain of transverse steel \(\varepsilon_{su}\) can also influence the factor \(\mu \phi\) of steel-confined columns. It should be noted that the ultimate strain of the transverse steel is one of the control parameters to determine the ultimate strain of steel-confined concrete, as shown in Eq.(5-6), which further influences the achievable ultimate curvature of column sections. For example, the ultimate strain of the US#3 steel spirals is \(\varepsilon_{su} = 0.17\) in the columns reported in Chapter 3. Thus, the computational results will be conservative for these specimens if taking \(\varepsilon_{su} = 0.1\).

Higher curvature ductility factor \(\mu \phi\) is resulted from higher ultimate strain of transverse confining steel \(\varepsilon_{su}\), which is shown by the comparison between the computational results in Figure 6.4(a) and (b). The computational data in Figure 6.4(a) are based on \(\varepsilon_{su} = 0.1\), while the data in Figure 6.4(b) are based on \(\varepsilon_{su} = 0.12\). Thus, the Eq.(6-2a) is further modified by the factor \(k_4\) as follows with good accuracy:

\[
\mu \phi = k_1 \cdot k_4 \cdot \frac{f_y}{(f'_c)^{1.5}} + k_2
\]

(6-2b)

where: the factors \(k_1\) to \(k_3\) are the same as for Eq.(6-2a), and \(k_4 = 10\varepsilon_{su}\). Considering the uncertainty of the fracture of the transverse confining steel in steel-confined columns, the value of \(\varepsilon_{su}\) in factor \(k_4\) is recommended to be limited by \(\varepsilon_{su} \leq 0.13\), the lower bound of Grade 400W steel bars (CAN/CSA-G30.18-09).
It should be mentioned that some columns with thick concrete cover can only achieve low curvature ductility when the level of confinement $\frac{f_l}{(f_c')^{1.5}}$ is low. This phenomenon is observed for columns with all $\rho m$ but is more noticeable for columns with $\rho m=0.1$, as observed in the first column of charts in Figure 6.3. It is because the flexural strength of these columns is
reduced significantly with the spalling of concrete cover, which leads to brittle column failure before the transverse confinement can effectively play its role. Such columns should be avoided if ductile behavior is required.

![Graph](image)

**Figure 6.4 Predicted results of \( \mu_\phi \) of steel-confined columns**

### 6.3.1.3 Simplified expressions

The interactive influence of \( k_p \) and \( \rho_l m \) on the development of \( \mu_\phi \) is reflected in Eq. (6-2b) with good accuracy. However, this expression may not be very convenient to be used in certain situations due to the interaction of parameters, thus the following simplified expression is proposed for the design purpose, in which the influences of variables \( k_p \) and \( \rho_l m \) on the \( \mu_\phi \) are taken into account individually without interaction.

Based on the parametric studies for each variable using the computational results, it is found that the composite parameter

\[
\alpha_s = \frac{f_i}{(f'c)^{1.5} \cdot (k_p)^{0.5}}
\]

can reflect the degree of influence of each individual variable of \( f_i \), \( f'c \) and \( k_p \) with the least scatter, thus it is used in the development of following expressions.

The data of \( \mu_\phi \) vs. \( \alpha_s \) relationship are presented in Figure 6.5(a) to (d) for steel-confined columns, categorized according to the mechanical ratio \( \rho_l m \) of longitudinal reinforcement. The ultimate strain of transverse confining steel is taken as \( \varepsilon_{su} = 0.1 \) in the computation of these columns. The effective data range presented in the following numerical regression is limited within \( \alpha_s = 0.01 \) to...
0.03 and \( \mu_\Phi < 40 \) for engineering practice. The centric trend line of data is displayed as the solid line in each figure, while the 5\(^{th}\) percentile lower bound of data is shown as the dashed line. The regressed expressions of the centerline and the 5\(^{th}\) percentile lower bound of data are all labeled in each figure, which lead to the following proposals.

Eq. (6-3) is proposed for the centerline of the \( \mu_\Phi \) vs. \( \alpha_s \) relationship, which can be used to predict the curvature ductility factor \( \mu_\Phi \) of steel-confined columns.

\[
\mu_\Phi = k_4 \cdot k_5 \cdot \alpha_s + k_6
\]  

(6-3)

where:

\[
\alpha_s = \frac{f_i}{(f_c')^{1.5} \cdot (k_p)^{0.5}}
\]

\[
k_5 = -1993 \rho \rho + 1386
\]

\[
k_6 = 15 \rho \rho - 3.5
\]

Furthermore, the 5\(^{th}\) percentile lower bound of \( \mu_\Phi \) for the steel-confined columns can be estimated by the following expression:

\[
\mu_\Phi = k_4 \cdot k_7 \cdot \alpha_s + k_8
\]  

(6-4)

where:

\[
k_7 = -2400 \rho \rho + 1500
\]

\[
k_8 = 26 \rho \rho - 9
\]

The influence of parameter \( \rho m \) on the curvature ductility factor \( \mu_\Phi \) of the steel-confined columns is clearly accounted for in the above two expression. Estimated by the center lines in Figure 6.5 for columns with \( \alpha_s = 0.02 \), the factor \( \mu_\Phi \) decreases from 22 to 14 with the increase of parameter \( \rho m \) from 0.1 to 0.4.

Lastly, the data in Figure 6.5(a) to (d) are collected together in Figure 6.6, which can clearly indicate the improvement of \( \mu_\Phi \) with the increasing parameter \( \alpha_s \). The centric trend line of data is displayed as the solid line, while the 5\(^{th}\) percentile lower bound of data is shown as the dashed
The center line can be represented by:

\[ \mu_\phi = 921k_4 \cdot \alpha_s \quad (6-5) \]

and the 5\textsuperscript{th} percentile lower bound is determined by:

\[ \mu_\phi = 665k_4 \cdot \alpha_s \quad (6-6) \]

In Eqs. (6-3) to (6-6), the factor \( k_4 \) is taken as \( k_4 = 10 \varepsilon_{ul} \leq 1.3 \), same as in Eq.(6-2b). It should be noted that the expressions (6-5) and (6-6) can only provide qualitative estimation of the \( \mu_\phi \) vs. \( \alpha_s \) relationship of steel-confined columns, since the results are not accurate enough for practical application due to the wide range of data distribution.

![Figure 6.5](image)

Figure 6.5  Simplified relationship between \( \mu_\phi \) and \( \alpha_s \) of steel-confined columns
6.3.2 FRP-confined columns

In this section, numerical analysis is conducted for the development of curvature ductility factor $\mu_\phi$ of the FRP-confined circular and square columns.

6.3.2.1 Parametric analysis

Based on the computational results, it is found that their seismic behaviour is not very sensitive to the diversity of FRP materials, i.e. CFRP or GFRP. It is also revealed that the effect of cross-sectional shape of the column on curvature ductility factor $\mu_\phi$ is correctly accounted for if the effective confining stress $f_l$ by FRP jackets is estimated appropriately. The area ratio of concrete cover to core, $(A_g/A_c)-1$, appears to have no significance influence on the $\mu_\phi$ either in the presence of FRP that wraps the entire section.

As reported in previous experimental research (Sheikh and Yau, 2002; Iacobucci, et al., 2003; Momen and Sheikh, 2005), the $\mu_\phi$ of FRP-confined columns is mainly controlled by the variables of $f_l$, $f_c'$ and $k_p$. It was observed that the $\mu_\phi$ increases with the increase of FRP confinement level, $f_l/f_c'$, and the decrease of the axial load level, $k_p$. However, it is revealed by the computational results that the $\mu_\phi$ vs. $k_p$ relationship of FRP-confined columns is also significantly influenced by the mechanical ratio $\rho_m$ of longitudinal reinforcement, similar to the steel-confined columns discussed in the previous section. As an example, the computational

\[ \mu_\phi = 921\alpha_s \]

\[ \mu_\phi = 665\alpha_s \]
results of FRP-confined circular columns of $f_c'=30$ MPa and $D=500$ mm are presented in Figure 6.7, in which the variations in the yield curvature $\phi_y$, the ultimate curvature $\phi_u$ and the curvature ductility factor $\mu_\phi$ as $k_p$ changes are shown for different values of $\rho m$. The following six points explain the data:

1) It is indicated by the first column of charts in Figure 6.7 that the relationship of $\phi_y$ vs. $k_p$ is significantly affected by $\rho m$. The increase of $k_p$ from 0.2 to 0.6 only results in a gentle decrease of $\phi_y$ for columns with $\rho m=0.1$, while for columns in which $\rho m=0.4$, the $\phi_y$ decreases sharply with the increase of $k_p$.

2) The first column of charts in Figure 6.7 also shows that the influence of $\rho m$ on $\phi_y$ is significantly affected by $k_p$. The $\phi_y$ increase from 6 to 9 with the increase of $\rho m$ from 0.1 to 0.4 for columns under $k_p=0.2$. But for columns under $k_p=0.6$, the influence of $\rho m$ on $\phi_y$ is minimal.

3) As shown in the second column of charts in Figure 6.7, the ultimate curvature $\phi_u$ always decreases with the increase of $k_p$ and the tendency is not significantly influenced by $\rho m$.

4) The charts in the second column of Figure 6.7 also indicate that the parameter $\rho m$ does not affect the $\phi_u$ to the same degree as the influence on $\phi_y$.

5) The relationship of $\mu_\phi$ vs. $k_p$ of FRP-confined columns is shown in the third column of charts in Figure 6.7. It can be seen that a rapid decrease of $\mu_\phi$ is resulted from the increase of $k_p$ from 0.2 to 0.6 for columns with $\rho m=0.1$, but the influence of $k_p$ on $\mu_\phi$ is not appreciable if $\rho m=0.4$. This phenomenon can be explained by the above-mentioned $\phi_y$ vs. $k_p$ relationship. For columns with $\rho m=0.4$, if the value of $k_p$ increases from 0.2 to 0.6, the decrease of $\phi_u$ is compensated by the decrease of $\phi_y$ in a similar degree and thus results in the almost unchanged $\mu_\phi$.

6) The influence $\rho m$ on $\mu_\phi$ is also clearly shown by the charts in the third column of Figure 6.7. For instance, for columns of $k_p=0.2$ and $f_/f_c=0.05$, the factor $\mu_\phi$ decreases from 20 to 10 with the increases of $\rho m$ from 0.1 to 0.4. But, if $k_p=0.6$, the $\mu_\phi$ only decreases slightly from 12 to 10. The interactive influence of parameters $k_p$ and $\rho m$ is indicated again by this phenomenon. In summary, transverse FRP-confinement can significantly improve the curvature ductility factor $\mu_\phi$. 
of columns. But the improvement of $\mu_{\phi}$ is influenced by parameters $k_p$ and $\rho m$ interactively. Considering all of these parameters is necessary to accurately predict the factor $\mu_{\phi}$.

![Figure 6.7 Influence of $k_p$ and $\rho m$ on $\mu_{\phi}$ of FRP-confined columns](image)

### 6.3.2.2 Proposed expressions

Based on the numerical analysis and comparison, it is found that the composite parameter $\frac{f_l'}{f_c'}$ can reflect the degree of influence of each variable of $f_l$ and $f_c'$ on the factor $\mu_{\phi}$ of FRP-confined
columns with the least scatter. The relationships between the composite parameters $\mu$ and $\frac{f_i}{f_c}$ are presented in Figure 6.8 by using the computational results of FRP-confined columns with square and circular cross sections, under the axial load level $k_p = 0.2$ to $0.6$, with concrete strength up to 60 MPa. For the purpose of engineering practice, the computational results of columns with impractically high level of $\mu$ and high confinement are excluded from the study, thus the effective data range is limited by $\mu < 30$ and $\frac{f_i}{f_c} < 0.15$.

In order to study the interactive influence of the axial load level $k_p$ and the mechanical ratio of longitudinal reinforcement $\rho_m$, the relationship of $\mu$ vs. $\frac{f_i}{f_c}$ is categorized according to parameters $k_p$ and $\rho_m$. The data in each column of charts in Figure 6.8 represent the computational results of columns of the same $\rho_m$, while the date in each row of charts in Figure 6.8 are the results of columns under the same $k_p$. The regressed centric trend line of data is labelled with its equation in each chart, which clearly indicates that the influences of $k_p$ and $\rho_m$ are interactive. Based on the equations of centric trend lines, following expression is proposed for the $\mu$ vs. $\frac{f_i}{f_c}$ relationship of FRP-confined columns:

$$
\mu = k_1 \cdot \left( \frac{f_i}{f_c} \right) + k_2
$$

(6-7)

where:

$$k_1 = 163(k_p)^{k_3}
$$

$$k_2 = 1.5(k_p)^{-0.75}
$$

$$k_3 = 1.72 \rho_m - 0.48
$$

The influence of $k_p$ depends on $\rho_m$, as indicated by the parameters $k_1$ and $k_3$. This expression represents most of the trend lines in Figure 6.8 with good accuracy ($R^2=0.93$), as shown in Figure 6.9 in which the abscissa is the computational results of $\mu$ by the program SAC, while the ordinate is the predicted results of $\mu$ using Eq.(6-7).
It should be mentioned that for FRP-confined columns with $\rho_m=0.1$ and low axial load, some results of $\mu_\phi$ show a large scatter. This is observed in the first row and first column of charts in Figure 6.98. It is because that the failure of some FRP-confined columns in this range is controlled by the fracture of longitudinal steel, rather than the failure of confined concrete.
6.3.2.3 Simplified expressions

The interactive influence of \( k_p \) and \( \rho m \) on the development of \( \mu_\phi \) is reflected in Eq. (6-7) with good accuracy. This expression may not be convenient to use in certain situations due to the interaction of parameters, thus the following simplified expression is proposed for the design purpose, in which the influences of \( k_p \) and \( \rho m \) are taken into account individually and without interaction.

Based on the parametric, it is observed that the composite parameter 
\[
\alpha_F = \frac{f_l}{f_c^{0.5}(k_p)}
\]

can reflect the degree of influence of each individual variable of \( f_l, f_c', \) and \( k_p \) with the least scatter, thus it is used in the development of following expressions.

The data of \( \mu_\phi \) vs. \( \alpha_F \) relationship are presented in Figure 6.10(a) to (d) for FRP-confined columns, categorized only according to the values of \( \rho m \). In each figure, the centric trend line of data is shown as a solid line, while the 5th percentile lower bound of data is displayed as a dashed line. The regressed expressions of the centerline and the 5th percentile lower bound of data are labeled in each figure.

Taking into account the effect of \( \rho m \), the following expression is proposed for the centric trend line of the \( \mu_\phi \) vs. \( \alpha_F \) relationship, which represents all the four centric trend lines in Figure 6.10:
\[ \mu_\phi = k_4 \cdot \alpha_F + 4.5 \]  \hspace{1cm} (6-8)

In a similar way, the expression of the 5th percentile lower bound of the \( \mu_\phi \) vs. \( \alpha_F \) relationship is proposed as:

\[ \mu_\phi = k_5 \cdot \alpha_F + 3.5 \]  \hspace{1cm} (6-9)

where:

\[ \alpha_F = \frac{f_i}{f_c \cdot (k_p)^{0.5}} \]

\[ k_4 = \frac{44}{(\rho m)^{0.5}} \]

\[ k_5 = \frac{35}{(\rho m)^{0.5}} \]

The influence of parameter \( \rho m \) on the curvature ductility factor \( \mu_\phi \) of the FRP-confined columns is clearly shown by the above two expressions and Figure 6.10. For example, the factor \( \mu_\phi \) decreases from 18 to 12 with the increase of parameter \( \rho m \) from 0.1 to 0.4 for FRP-confined columns with \( \alpha_F = 0.1 \), estimated by the centric trend lines in Figure 6.10.

Lastly, the data in Figure 6.10(a) to (d) are collected together in Figure 6.11, which indicate the improvement of the factor \( \mu_\phi \) with the increasing parameter \( \alpha_F \). The centric trend line of data is displayed as the solid line, while the 5th percentile lower bound of data is shown as the dashed line. The center line can be represented by

\[ \mu_\phi = 98 \cdot \alpha_F + 4.5 \]  \hspace{1cm} (6-10)

and the 5th percentile lower bound can be estimated by:

\[ \mu_\phi = 65 \cdot \alpha_F + 3.5 \]  \hspace{1cm} (6-11)

It should be noted that the expressions (6-10) and (6-11) can provide only qualitative estimation of the \( \mu_\phi \) vs. \( \alpha_F \) relationship of FRP-confined columns. The results are not accurate enough for practical application due to the wide range of data distribution, as shown in Figure 6.11.
Figure 6.10 Simplified relationship between $\mu_\phi$ and $\alpha_F$ of FRP-confined columns

Figure 6.11 Qualitative relationship between $\mu_\phi$ and $\alpha_F$ of FRP-confined columns
6.4 Displacement ductility factor

6.4.1 Steel-confined columns

Numerical analysis is conducted in this section for the development of the displacement ductility factor $\mu_\Delta$ of steel-confined circular and square columns based on the computational results by using the program SAC.

6.4.1.1 Parametric analysis

Unlike the curvature ductility factor $\mu_\varphi$ studied in the previous sections, it has been observed in experimental research and numerical analysis that the displacement ductility factor $\mu_\Delta$ of steel-confined columns is not only significantly influenced by the amount of effective transverse confinement $f_l$, concrete strength $f'_c$, the axial load level $k_p$ and the mechanical ratio of longitudinal reinforcement $\rho_m$, but also by the shear span-depth ratio $\lambda$ of columns and the area ratio $A_g/A_{cov}$, where $A_g$ = gross cross-sectional area of the column; $A_{cov}$ = cross-sectional area of concrete cover, measured to the centerline of the peripheral transverse steel. However, the effect of the cross-sectional shape of steel-confined columns on $\mu_\Delta$ is correctly accounted for if the effective confining stress $f_l$ is estimated appropriately, similar to the observation made for the curvature ductility factor $\mu_\varphi$.

Based on the comparison of data, it is found that the composite parameter \( \frac{f_l}{(f'_c)^{1.5}} \cdot \left( \frac{A_g}{A_{cov}} \right) \) reflects the influence of each variable of $f_l$, $f'_c$, and $A_g/A_{cov}$ on the development of $\mu_\Delta$ with the least scatter for steel-confined columns. Thus, the relationships between $\mu_\Delta$ and $\frac{f_l}{(f'_c)^{1.5}} \cdot \left( \frac{A_g}{A_{cov}} \right)$ is presented in Figure 6.12 to Figure 6.14. The data on each individual chart represents the computational results of columns with the same values of $\lambda$, $k_p$, and $\rho_m$. The trend line of data is labelled in each chart with its equation. For the purpose of engineering practice, the computational results of columns with impractically high level of $\mu_\Delta$ and high confinement are excluded from the study, thus the effective data range is limited by $\mu_\Delta < 15$. Following observations are made from the charts in Figure 6.12 to Figure 6.14:
1) The factor $\mu_\Delta$ of steel-confined columns decreases significantly with the increase of the shear span-depth ratio $\lambda$ of columns. In most situations, $\mu_\Delta$ may decrease to about 50% if the $\lambda$ increases from 3 to 5, as demonstrated by the trend lines of the data. For example, for steel-confined columns of $\frac{f_i}{(f'_c)^{1/3}}, \left(\frac{A_g}{A_{cov}}\right)=0.05$, $k_p =0.3$ and $\rho_m=0.3$, the center lines of data indicate that the $\mu_\Delta$ of columns may decrease from 8.6 to 4.3 with the increase of $\lambda$ from 3 to 5. This phenomenon was also reported in a fib state-of-the-art report, in which it was concluded empirically that the shear span-depth ratio was the most important parameter for the member deformability (fib Task Group 7.2, 2003). Thus, the influence of parameter $\lambda$ cannot be ignored and should be taken into account in the study of $\mu_\Delta$.

2) The factor $\mu_\Delta$ of steel-confined columns decreases with the increase of axial load level $k_p$. But, the degree of this influence varies with the $\rho_m$ of columns. For example, for steel-confined columns of $\frac{f_i}{(f'_c)^{1/3}}, \left(\frac{A_g}{A_{cov}}\right)=0.05$, $\lambda =4$ and $\rho_m=0.1$, the $\mu_\Delta$ decreases from 6.5 to 5.1 with the increase of $k_p$ from 0.2 to 0.6, according to the center lines of data in the charts. But, if the same column has $\rho_m=0.4$, the $\mu_\Delta$ decreases from 6.7 to 5.8 only with the same increase of $k_p$. Obviously, the influence of $k_p$ on the $\mu_\Delta$ is more significant for columns with lower $\rho_m$. Similar phenomenon of the interactive influenced of $k_p$ and $\rho_m$ was also observed in the study of curvature ductility factor $\mu_\phi$ of steel-confined columns.

3) The factor $\mu_\Delta$ of steel-confined columns increases with the increase of the mechanical ratio of longitudinal reinforcement $\rho_m$ in most situations. For most columns of $\lambda =4$ and 5 and subjected to high axial loads, the $\mu_\Delta$ may increase for about 10% to 20% with the increase of $\rho_m$ from 0.1 to 0.4, estimated by the center lines of data in the charts. It should be noted that the influence of $\rho_m$ on $\mu_\Delta$ is opposite to its influence on $\mu_\phi$ of steel-confined columns. Recall that, reported in a previous section for an example column of $k_p =0.2$ and $f_i/f'_c = 0.05$, the factor $\mu_\phi$ decreases from 20 to 10 if the parameter $\rho_m$ increases from 0.1 to 0.4. It appears that the slope of the descending branches of lateral shear vs. tip deflection curves is lessened with the increase of $\rho_m$, which results in the increase of $\mu_\Delta$. 
Furthermore, in order to understand the interactive influences of $k_p$ and $\rho_m$ more clearly, a steel-confined circular column of $\lambda = 3, f_c' = 30$ MPa, $D = 500$ mm and $cc = 30$ mm is used as an example. The computational results of this column are provided in Figure 6.15, in which the variations in the yield displacement $\Delta_y$, the ultimate displacement $\Delta_u$ and the displacement ductility factor $\mu_\lambda$ with the change of $k_p$ are shown for different values of $\rho_m$. Following points can be made from this analysis:

1) It is indicated by the first column of charts in Figure 6.15 that the relationship of $\Delta_y$ vs. $k_p$ is significantly affected by $\rho_m$. The increase of $k_p$ from 0.2 to 0.6 only results in a slight decrease of $\Delta_y$ for columns of $\rho_m=0.1$, while the $\Delta_y$ decreases rapidly for columns of $\rho_m=0.4$.

2) In the second column of charts in Figure 6.15, the relationship of $\Delta_u$ vs. $k_p$ is also significantly affected by $\rho_m$. A gentle decrease of $\Delta_u$ for columns of $\rho_m=0.1$ is resulted by the increase of $k_p$ from 0.2 to 0.6, while this decrease of $\Delta_u$ is much rapider for columns of $\rho_m=0.4$.

3) The relationship of $\mu_\lambda$ vs. $k_p$ of the steel-confined column is presented in the third column of charts in Figure 6.15. It is seen that influences of $\rho_m$ and $k_p$ are not very significant, which can be explained by the above-mentioned $\Delta_y$ vs. $k_p$ and $\Delta_u$ vs. $k_p$ relationships. For columns with $\rho_m=0.1$ to 0.4, the $\Delta_y$ and $\Delta_u$ decrease to a similar degree with the increase of $k_p$ from 0.2 to 0.6. Thus, the decrease of the resulting displacement ductility factor, $\mu_\lambda = \Delta_u / \Delta_y$, is not so dramatic.

In summary, the $\mu_\lambda$ of steel-confined columns is significantly influenced by the variable $\lambda$ which should be taken into account in the development of $\mu_\lambda$. The variables $k_p$ and $\rho_m$ may also interactively influence the development of $\mu_\lambda$ but to a lesser degree, thus the consideration of $k_p$ and $\rho_m$ can further increase the accuracy of the prediction of $\mu_\lambda$. 
Figure 6.12 Influence of variables on $\mu_\Delta$ of steel-confined columns of $\lambda = 3$

$\rho_m = 0.1$
kp = 0.2

$\rho_m = 0.2$
kp = 0.2

$\rho_m = 0.3$
kp = 0.2

$\rho_m = 0.4$
kp = 0.2

$\rho_m = 0.1$
kp = 0.3

$\rho_m = 0.2$
kp = 0.3

$\rho_m = 0.3$
kp = 0.3

$\rho_m = 0.4$
kp = 0.3

$\rho_m = 0.1$
kp = 0.4

$\rho_m = 0.2$
kp = 0.4

$\rho_m = 0.3$
kp = 0.4

$\rho_m = 0.4$
kp = 0.4

$\rho_m = 0.1$
kp = 0.5

$\rho_m = 0.2$
kp = 0.5

$\rho_m = 0.3$
kp = 0.5

$\rho_m = 0.4$
kp = 0.5

$\rho_m = 0.1$
kp = 0.6

$\rho_m = 0.2$
kp = 0.6

$\rho_m = 0.3$
kp = 0.6

$\rho_m = 0.4$
kp = 0.6

$y = 135.8x + 3.4$

$y = 127.4x + 3.4$

$y = 122.1x + 3.4$

$y = 121.5x + 3.4$

$y = 112.5x + 3.0$

$y = 111.6x + 3.0$

$y = 112.7x + 3.0$

$y = 111.4x + 3.0$

$y = 106.3x + 2.6$

$y = 118.5x + 2.6$

$y = 123.9x + 2.6$

$y = 121.2x + 2.6$

$y = 116.3x + 2.2$

$y = 112.3x + 2.2$

$y = 125.6x + 2.2$

$y = 123.9x + 2.2$

$y = 129.0x + 2.2$

$y = 115.0x + 1.7$

$y = 126.5x + 1.7$

$y = 130.7x + 1.7$

$y = 129.0x + 1.7$
Figure 6.13 Influence of variables on $\mu_\Delta$ of steel-confined columns of $\lambda = 4$
Figure 6.14 Influence of variables on $\mu_\Delta$ of steel-confined columns of $\lambda = 5$
6.4.1.2 Proposed expressions

Combining all the expressions of centric trend lines of the computational data in Figure 6.12 to Figure 6.14, following expression is proposed to predict the development of the factor $\mu_{\Delta}$ of steel-confined columns:

$$\mu_{\Delta} = k_1 \cdot \frac{f_i}{f_{c'}^{0.15}} \cdot \left( \frac{A_g}{A_{cov}} \right) + k_2$$

(6-12)
where:

\[ k_1 = k_3 \cdot k_p + k_4 \]

\[ k_2 = \left( \frac{6}{\lambda} + 2.3 \right) \cdot (1 - k_p) \]

\[ k_3 = 90 \cdot (\rho m) + 35 \]

\[ k_4 = \frac{573}{\lambda} - 97 \]

The above expression represents most of the trend lines in Figure 6.12 to Figure 6.14 with the least error. The accuracy of Eq.(6-12) is shown in Figure 6.4 for steel-confined columns, in which the abscissa is the computational results of \( \mu_\Delta \) by the program SAC, while the ordinate is the predicted results of \( \mu_\Delta \) using Eq.(6-12).

![Figure 6.16 Predicted results of \( \mu_\Delta \) of steel-confined columns](image)

6.4.1.3 Simplified expressions

The interactive influence of variables \( \lambda, k_p \), and \( \rho m \) on the development of \( \mu_\Delta \) is reflected in Eq. (6-12) by the parameters \( k_1 \) to \( k_4 \) with good accuracy. But, this expression may not be convenient for use in engineering practice where less laborious estimation is needed with reasonable accuracy. Thus a simplified expression is proposed below for the design purpose, in which the influences variables \( \lambda \) and \( k_p \) are represented by parameters independently, while the effect of \( \rho m \) on \( \mu_\Delta \) is ignored.
It has been found that the composite parameter \( \beta_s = \frac{f_l}{(f_c^*)^{1.5} (k_p)^{0.5} \left( \frac{A_g}{A_{cov}} \right)} \) can reflect the influence of each variable on the development of \( \mu_\Delta \) with the least scatter. Thus, this parameter is used to derive the simplified expressions of \( \mu_\Delta \) for the steel-confined columns. The computational results of the \( \mu_\Delta \) vs. \( \beta \) relationship are categorized according to different values of \( \lambda \) in Figure 6.17(a) to (c). In each figure, the centric trend line of data is displayed as the solid line, while the dashed line is the 5th percentile lower bound. The regressed linear expressions are labeled in each figure for the centric trend line and the 5th percentile lower bound.

Combining the three cases in Figure 6.17(a) to (c), the following expression is proposed for the centric trend line as:

\[
\mu_\Delta = k_s \cdot \beta_s + 2.0 \quad (6-13)
\]

and the 5th percentile lower bound of data is determined by:

\[
\mu_\Delta = k_6 \cdot \beta_s + 1.1 \quad (6-14)
\]

where:

\[
\beta_s = \frac{f_l}{(f_c^*)^{1.5} (k_p)^{0.5} \left( \frac{A_g}{A_{cov}} \right)}
\]

\[
k_s = \frac{376}{\lambda} - 46
\]

\[
k_6 = \frac{292}{\lambda} - 30
\]

As an example, the required confinement for the steel-confined concrete column with \( \lambda = 4 \) is presented here. According to the equation of the 5th percentile lower bound, \( \beta_s \geq 0.067 \) is required for the ductile column of \( \mu_\Delta \geq 4.0 \), while required \( \beta_s \geq 0.033 \) for the moderately ductile column of \( \mu_\Delta \geq 2.5 \).

It is noticed that the displacement ductility is influenced by more variables than the curvature ductility for steel-confined columns, and the data in numerical study has larger scatter. Thus, the 5th percentile lower bound is proposed for design rather than the centric trend line. Based on the expressions in Figure 6.17, it can be seen that the displacement ductility factor determined by the
centric trendlines is about 20-40% higher than that by the expressions of the lower bounds for steel-confined columns with the same value of $\beta_s$. This indicates the extra margin of safety for the expressions of the lower bounds.

![Graphs showing relationship between $\mu_\Delta$ and $\beta_s$ for different values of $\lambda$.](image)

Figure 6.17 Simplified relationship between $\mu_\Delta$ and $\beta_s$ of steel-confined columns

### 6.4.2 FRP-confined columns

In this section, numerical analysis is conducted for the development of displacement ductility factor $\mu_\Delta$ of FRP-confined circular and square columns based on the computational results by using the program SAC.

#### 6.4.2.1 Parametric analysis

The parametric analysis reveals that the development of $\mu_\Delta$ of the FRP-confined columns is influenced by the variables $f_i$, $f'_c$, $\lambda$, $k_p$ and $\rho m$, while the effect of the cross-sectional shapes of
columns on the factor $\mu_\Delta$ is correctly accounted for if the effective confining stress $f_i$ is estimated appropriately, similar to the observation of the factor $\mu_\phi$ of FRP-confined columns. The area ratio of the concrete core within steel cage to the gross cross section of the column was found to have only negligible influence on the factor $\mu_\Delta$ of FRP-confined columns since the whole cross section is confined by FRP jackets, which is different from the steel-confined columns.

The influences of the shear span-depth ratio $\lambda$, the axial load level $k_p$ and the mechanical ratio of longitudinal reinforcement $\rho_m$ on the displacement ductility factor $\mu_\Delta$ of FRP-confined columns are illustrated by the charts in Figure 6.18 to Figure 6.20, in which the abscissa is the composite parameter $\frac{f_i}{(f_c')^{1.5}}$. The data shows minimal scatter. Each individual chart represents the computational results of the factor $\mu_\Delta$ factors of columns with the same values of $\lambda$, $k_p$ and $\rho_m$. The linear trend line of data with its expression is also labelled on each chart. Following points can be made for the FRP-confined columns from the comparison:

1) The factor $\mu_\Delta$ decreases significantly with the increase of the shear span-depth ratio $\lambda$ of columns. In most situations, $\mu_\Delta$ may decrease to about 50% if the $\lambda$ increases from 3 to 5, reflected by the trend lines of data. For example, for columns of $\frac{f_i}{(f_c')^{1.5}}=0.01$, $k_p=0.3$ and $\rho_m=0.3$, the centerlines of data indicate that the $\mu_\Delta$ of FRP-confined columns decreases from 11.3 to 6.3 with the increase of $\lambda$ from 3 to 5. This phenomenon was very similar to that of steel-confined columns.

2) The influence of the mechanical ratio of longitudinal reinforcement $\rho_m$ on the factor $\mu_\Delta$ differs with the different $\lambda$ values, as shown by the centerlines of data. The factor $\mu_\Delta$ of FRP-confined columns of $\lambda=3$ decreases with the increase of the $\rho_m$ in most situations. However, for most columns of $\lambda=4$ and 5, the influence of $\rho_m$ is only minimal. In some situations, such as for columns of $\lambda=5$ and $k_p \geq 0.4$, the factor $\mu_\Delta$ increases slightly with the increase of $\rho_m$ from 0.1 to 0.4. This phenomenon was also found in the study of $\mu_\Delta$ of steel-confined columns.

3) The influence of the axial load level $k_p$ on the factor $\mu_\Delta$ of FRP-confined columns depends on the $\rho_m$ of columns, reflected by the centerlines of data. For example, for columns of $\frac{f_i}{(f_c')^{1.5}}$
=0.01, \( \lambda =4 \) and \( \rho m=0.1 \), the \( \mu_\Delta \) decreases from 12.6 to 7.9 with the increase of \( k_p \) from 0.2 to 0.6. While, for the same column but \( \rho m=0.4 \), the \( \mu_\Delta \) decreases from 8.7 to 8.0 with the same increase of \( k_p \). It is indicated that the influence of \( k_p \) on the \( \mu_\Delta \) is more significant for columns with lower \( \rho m \), similar to the phenomenon of the interactive influences of \( k_p \) and \( \rho m \) found in the study of \( \mu_\Delta \) of steel-confined columns.

Furthermore, in order to understand the interactive influences of \( k_p \) and \( \rho m \) more clearly, an FRP-confined circular column of \( \lambda =3, f'_c=30 \) MPa and \( D=500 \) mm is used as an example. The computational results of this column are provided in Figure 6.21, in which the variations in the yield displacement \( \Delta_y \), the ultimate displacement \( \Delta_u \) and the displacement ductility factor \( \mu_\Delta \) with the change of \( k_p \) are shown for different values of \( \rho m \). Following points can be made:

1) It is indicated by the first column of charts in Figure 6.21 that the relationship of \( \Delta_y \) vs. \( k_p \) is significantly affected by \( \rho m \). The increase of \( k_p \) from 0.2 to 0.6 only results in a minimal decrease of \( \Delta_y \) for columns of \( \rho m=0.1 \). On the contrary, for columns of \( \rho m=0.4 \) the \( \phi_y \) decreases rapidly from 7.7 mm to 4.4 mm with the increase of \( k_p \) from 0.2 to 0.6.

2) Shown in the second column of charts in Figure 6.21, the relationship of \( \Delta_u \) vs. \( k_p \) is not significantly affected by \( \rho m \). The increase of \( k_p \) from 0.2 to 0.6 always results in the decrease of \( \Delta_u \) in a similar tendency.

3) The relationship of \( \mu_\Delta \) vs. \( k_p \) of the FRP-confined column is presented in the third column of charts in Figure 6.21, in which the interactive influences of \( \rho m \) and \( k_p \) are shown clearly. For example, for columns with \( \rho m=0.1 \) and \( f'/f_c=0.02 \), the \( \mu_\Delta \) decreases from 11 to 7 with the increase of \( k_p \) from 0.2 to 0.6. Meanwhile, the \( \mu_\Delta \) increases from 5 to 6 for the same column but \( \rho m=0.4 \).

In summary, the \( \mu_\Delta \) of FRP-confined columns is significantly influenced by the variable \( \lambda \) which should be taken into account in the development of \( \mu_\Delta \). The variables \( k_p \) and \( \rho m \) also influence \( \mu_\Delta \) interactively to a certain degree, thus the consideration of \( k_p \) and \( \rho m \) can further increase the accuracy of the prediction of \( \mu_\Delta \).
Figure 6.18 Influence of variables on $\mu_{\Delta}$ of FRP-confined columns of $\lambda = 3$
Figure 6.19 Influence of variables on $\mu_\Delta$ of FRP-confined columns of $\lambda = 4$
Figure 6.20 Influence of variables on $\mu_\Lambda$ of FRP-confined columns of $\lambda = 5$
6.4.2.2 Proposed expressions

Based on the preceding discussion and computational results in Figure 6.18 to Figure 6.20, the following expression is proposed to represent the centerlines in the charts with the least error:

\[
\mu_\Delta = k_1 \cdot \frac{f_t}{\left(f'_c\right)^{1.5}} + k_2
\]  

(6-15)
where:

\[ k_1 = k_1 \cdot k_p + k_4 \]

\[ k_2 = \left( \frac{4}{\lambda} + 3.5 \right) \cdot (1 - k_p) \]

\[ k_3 = \left( \frac{1504 \times 10^3}{\lambda^5} - 322 \right) \cdot (\rho, m) + \left( \frac{485 - 480 \times 10^3}{\lambda^5} \right) \]

\[ k_4 = \left( 410 - \frac{962 \times 10^3}{\lambda^5} \right) \cdot (\rho, m) + \left( \frac{418 \times 10^3}{\lambda^5} + 110 \right) \]

The influences of variables \( \lambda \), \( k_p \) and \( \rho m \) are interactive, indicated by the parameters \( k_1 \) to \( k_4 \) in the above expression. Figure 6.22 shows a comparison between the computational results of \( \mu_\Delta \) by the program SAC and the predicted results of \( \mu_\Delta \) using the above expression. It is noted that the data in Figure 6.22 is still distributed in a rather wide range even after the consideration of the complex interaction among the influences of variables \( \lambda \), \( k_p \) and \( \rho m \). Other variables such as the strength of concrete, the distribution of longitudinal steel and the shapes of columns also contribute to the scatter of data, even though each of them may have only minimal effect. Furthermore, this equation may not be not convenient for use in engineering practice. Thus, a simplified but conservative method is presented in the next section for a quick estimation of the \( \mu_\Delta \) of FRP-confined columns.

![Figure 6.22 Predicted results of \( \mu_\Delta \) of FRP-confined columns](image)
6.4.2.3 Simplified expressions

Based on the computational results, it is found that the influence of \( f_i, f'_c \) and \( k_p \) on the development of \( \mu_\Delta \) can be considered accurately by the composite parameter \( \beta_F = \frac{f_i}{(f'_c)^{1.5} (k_p)^{0.5}} \).

Thus, this parameter is used to derive the simplified but conservative expressions of \( \mu_\Delta \) for FRP-confined columns. The computational results of the \( \mu_\Delta \) vs. \( \beta_F \) relationship of FRP-confined columns are presented in Figure 6.23(a) to (c) for different values of \( \lambda \). In each figure, the centric trend line of data is displayed as the solid line, while the dashed line is the 5\(^{th}\) percentile lower bound. The regressed linear expressions are labeled in each figure for the centric trend line and the 5\(^{th}\) percentile lower bound.

Combining the expressions in Figure 6.23(a) to (c), the following equation is proposed for the centric trend line as:

\[
\mu_\Delta = k_5 \cdot \beta_F + 2.0
\]  

(6-16)

Similarly, the 5\(^{th}\) percentile lower bound can be represented by:

\[
\mu_\Delta = k_6 \cdot \beta_F + 1.3
\]  

(6-17)

where:

\[
\beta_F = \frac{f_i}{(f'_c)^{1.5} (k_p)^{0.5}}
\]

\[
k_5 = 1098-163 \lambda
\]

\[
k_6 = 730-100 \lambda
\]

As an example, according to Eq.(6-17) which is the 5\(^{th}\) percentile lower bound of the displacement ductility factor \( \mu_\Delta \), the required confinement is \( \beta_F \geq 0.0082 \) for the FRP-confined ductile concrete column with \( \lambda = 4 \) and the target ductility \( \mu_\Delta \geq 4.0 \). If the target displacement ductility \( \mu_\Delta \geq 2.5 \), then the required confinement of this column is \( \beta_F \geq 0.0036 \).

As discussed earlier for steel-confined columns, the displacement ductility is influenced by more variables than the curvature ductility for FRP-confined columns, and the numerical data has
larger scatter. Thus, the 5th percentile lower bound is proposed for design rather than the centric trend line. Figure 6.23 shows that the displacement ductility factor determined by the centric trendlines is about 30-40% higher than that by the expressions of the lower bounds for FRP-confined columns with the same value of $\beta_F$.

Figure 6.23 Simplified relationship between $\mu_\Delta$ and $\beta_F$ of FRP-confined columns
6.5 Lateral drift ratio

6.5.1 Steel-confined columns

Numerical analysis is conducted in this section for the development of the lateral drift ratio $\delta$ of steel-confined circular and square columns based on the computational results from the program SAC.

6.5.1.1 Parametric analysis

In the computational results, it was observed that the development of the drift ratio $\delta$ of steel-confined columns is not only significantly influenced by the amount of effective transverse confinement $f_l$, the concrete strength $f_c'$ and the axial load level $k_p$, but also affected dramatically by the area ratio of the concrete cover to concrete core of columns, $\frac{A_c}{A_c} - 1$. It is found that the composite parameter $\gamma_s = \frac{f_l}{f_c' \cdot k_p \cdot \left(\frac{A_g}{A_c} - 1\right)}$ can reflect the influence of individual variables with the least scatter. Thus, the parameter $\gamma_s$ is used in the development of expressions for the drift ratio $\delta$ of steel-confined columns. The effect of cross-sectional shape of columns on the ratio $\delta$ is correctly accounted for through appropriate use of effective confining stress $f_l$.

Recall that, during the study of the factor $\mu_\Delta$ for steel-confined columns, it was observed that the ultimate displacement $\Delta_u$ of steel-confined columns is significantly influenced by the mechanical ratio of longitudinal reinforcement, $\rho_m$, which was shown by an example in the second column of charts in Figure 6.15. For instance, in the case of $f_l/f_c' = 0.05$ and $k_p = 0.3$ in Figure 6.15, the $\Delta_u$ increases from 22 mm to 45 mm with the increase of $\rho_m$ from 0.1 to 0.4. Thus, the influence of $\rho_m$ is considered in the study of the drift ratio $\delta$. Based on the comparison of the computational results of all the steel-confined columns with different values of $\rho_m$, it is found that the factor $k_1 = 0.75 + 1.1 \rho_m$ can reflect the influence of $\rho_m$ on the $\delta$ of most columns with the least error and will be used in the following study. It is also noted that $\rho_m$ has larger influence on the drift ratio $\delta$ than on the displacement ductility factor $\mu_\Delta$ of steel-confined
columns. This is because the factor $\mu_\Delta$ is defined as the ratio of two variables, i.e. the ultimate displacement $\Delta_u$ and the yield displacement $\Delta_y$. Both of them increase with the value of $\rho m$, thus the influence of $\rho m$ on the factor $\mu_\Delta$ is not as significant as that on the drift ratio $\delta$.

The computational results of the drift ratio $\delta$ of steel-confined columns are shown in Figure 6.24, which represents the computational results of the steel-confined circular and square columns with concrete strength $f'_c = 30$ MPa to 60 MPa, under the axial load level $k_p = 0.2$ to 0.6. For the purpose of engineering practice, the columns with unnecessary high confinement are excluded by limiting the effective data within the range of $\gamma_s \leq 1.0$. The term $k_2$ in Figure 6.24(a) is a factor to allow for the influence of the shear span-depth ratio $\lambda$ of columns, which will be discussed later.

Figure 6.24  Computational results of drift ratio $\delta$ of steel-confined columns
The computational results are summarized for all the values of $\lambda$ in Figure 6.24(a), while the data is categorized in Figure 6.24(b) to (d) according to the different values of $\lambda$. The centric trend line of data is displayed as the solid line while the 5th percentile lower bound of data is shown as the dashed line, with regressed equations labeled in each figure. Similar to the displacement ductility factor $\mu_\Delta$, the drift ratio $\delta$ of steel-confined columns also significantly decreases with shear span-depth ratio $\lambda$ of columns. Thus, the parameters $\gamma_s$, $\rho m$ and $\lambda$ are taken into account in the following study for the development of the drift ratio $\delta$ of steel-confined columns.

6.5.1.2 Proposed expressions

The following expression is proposed based on the centerlines of data in Figure 6.24(b) to (d), which can be used to estimate the lateral drift ratio $\delta$ of steel-confined columns:

$$\delta = 0.033k_1 \cdot k_2 \cdot \sqrt{\gamma_s} - 0.05$$

(6-18)

The 5th percentile lower bound of data in Figure 6.24(b) to (d) is represented by the following equation:

$$\delta = 0.027k_1 \cdot k_2 \cdot \sqrt{\gamma_s} - 0.05$$

(6-19)

where:

$$\gamma_s = \frac{f_i}{f_c' \cdot k_p \cdot \left(\frac{A_g}{A_c} - 1\right)}$$

is a composite parameter; $k_1 = 0.75 + 1.1\rho_m$, is the factor to allow for the influence of the mechanical ratio $\rho_m$ of longitudinal reinforcement; $k_2 = 1.55 - 0.135\lambda$, is the factor to allow for the influence of the shear span-depth ratio $\lambda$ of columns.

Eq.(6-18) proposed for the center lines of data can predict the drift ratio $\delta$ of steel-confine columns with pretty good accuracy ($R^2=0.935$), as shown in Figure 6.24(a). Meanwhile, the Eq.(6-19) can be used when a conservative estimation of $\delta$ is preferred considering the range of data distribution. It is noted that the estimation of $\delta$ by using Eq.(6-19) is about 0.83 of that by the Eq.(6-18) of the centerlines.
In the above two expressions, the value of $k_1$ increases from 0.86 to 1.19 with the increase of $\rho_m$ from 0.1 to 0.4 while the value of $k_2$ decreases from 1.15 to 0.88 with the increase of $\lambda$ from 3 to 5, which reflect the degree of the influences of these two variables. The influences of the variables $\rho_m$ and $\lambda$ can further be shown more clearly by the following example. The drift ratio $\delta$ is estimated for a steel-confined concrete column with the composite parameter $\gamma_s = 0.5$ by using Eq.(6-18). As shown in Table 6.1, the estimated value of $\delta$ varies between 1.7% and 3.0% due to the variation of variables $\rho_m$ and $\lambda$ within the ranges of $\rho_m = 0.1$ to 0.4 and $\lambda = 3$ to 5. This example indicates that the ratio $\delta$ of steel-confined columns is significantly influenced by the combined effects of variables $\rho_m$ and $\lambda$, even though the degree of influence by either of them is not so large. In the absence of the detailed information about $\rho_m$ or $\lambda$, the drift ratio of columns can be estimated approximately by taking the corresponding factor $k_1$ or $k_2$ as 1.0. However, the resulting error should be recognized.

<table>
<thead>
<tr>
<th>$\lambda$</th>
<th>$\delta$ (%)</th>
<th>$\rho_m$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.1</td>
<td>0.2</td>
</tr>
<tr>
<td>3</td>
<td>2.2</td>
<td>2.5</td>
</tr>
<tr>
<td>4</td>
<td>1.9</td>
<td>2.2</td>
</tr>
<tr>
<td>5</td>
<td>1.7</td>
<td>1.9</td>
</tr>
</tbody>
</table>

6.5.2 FRP-confined columns

Numerical study is conducted in this section for the development of the lateral drift ratio $\delta$ of FRP-confined circular and square columns based on the computational results by using the program SAC.

6.5.2.1 Parametric analysis

It has been observed that the drift ratio $\delta$ of FRP-confined columns is mainly influenced by the amount of effective transverse confinement $f_t$, the concrete strength $f_{ct}$ and the axial load level $k_p$. The effect of the cross-sectional shapes of columns on the ratio $\delta$ is correctly accounted for if the effective confining stress $f_t$ is estimated appropriately, similarly to the observations made for the
factors $\mu_\phi$ and $\mu_\Lambda$ of FRP-confined columns. The area ratio of concrete cover to core of columns, $\frac{A_g}{A_c} - 1$, has only insignificant influence on the $\delta$ of FRP-confined columns since the whole cross section is confined by FRP jackets, which is different from the comparable steel-confined columns. Based on the comparison of computational data, it is found that the composite parameter $\gamma_F = \frac{f_i}{f_c^{\prime} \cdot k_p}$ can reflect the degree of influence of each variable with the least scatter, thus it is used in the following study for the drift ratio $\delta$ of FRP-confined columns.

During the study of the displacement ductility factor $\mu_\Lambda$ of FRP-confined columns in a previous section, it was found that the ultimate displacement $\Delta_u$ is influenced by the mechanical ratio of longitudinal reinforcement $\rho_{lm}$, which was shown by an example in the charts of the second column in Figure 6.21. Under the conditions of $f_i/f_c^{\prime} = 0.05$, $\lambda = 3$ and $k_p = 0.3$ in that example, the $\Delta_u$ varies between 54 mm and 62 mm with the increase of $\rho_{lm}$ from 0.1 to 0.4. Thus, the accuracy of the predicted $\delta$ can be improved by considering the influence of $\rho_{lm}$. The computational results also show that the factor $k_1 = 1.05 - 5.7(0.4 - \rho_{lm})^3$ can reflect the influence of $\rho_{lm}$ on the $\delta$ with the least error for most FRP-confined columns and will be used in the following study.

The computational results of the drift ratio $\delta$ are shown in Figure 6.25, which include the FRP-confined circular and square columns with concrete strength $f_c^{\prime}$ = 30 MPa to 60 MPa, subjected to axial load level $k_p$ = 0.2 to 0.6. For the purpose of engineering practice, the columns with unnecessary high confinement are excluded from the following study by limiting the effective data as $\gamma_F \leq 0.4$. The term $k_2$ in Figure 6.25(a) is a factor to allow for the influence of the shear span-depth ratio $\lambda$ of columns, which will be discussed later.

The computational results of columns with different values of $\lambda$ are collected together in Figure 6.25(a), while the data in Figure 6.25(b) to (d) is categorized according to the value of $\lambda$. The centric trend line of data is displayed as a solid line while the 5th percentile lower bound of data as a dashed line with the regressed equations in each chart. The relationship of $\delta$ vs. $\gamma_F$ of FRP-confined columns is also significantly influenced by the shear span-depth ratio $\lambda$ of columns which is shown clearly in Figure 6.25(b) to (d), similar as the situation of displacement ductility.
factor $\mu_\lambda$. Thus, the parameters $\gamma_F, \rho_m$ and $\lambda$ are taken into account in the following study for the development of the drift ratio $\delta$ of FRP-confined columns.

Figure 6.25 Computational results of drift ratio $\delta$ of FRP-confined columns

6.5.2.2 Proposed expressions

The following expression is proposed to represent the three centerlines of data in Figure 6.25(b) to (d), which can be used to estimate the drift ratio $\delta$ of FRP-confined columns:

$$\delta = 0.099 k_1 k_2 (\gamma_F - 0.015)^{0.5}$$

(6-20)
The 5th percentile lower bound of data in Figure 6.25(b) to (d) are represented by the following expression:

$$\delta = 0.079 k_1 \cdot k_2 \cdot \sqrt{\gamma_F} - 0.015$$  \hspace{1cm} (6-21)

where:

$$\gamma_F = \frac{f_l}{f_c \cdot k_p}, \text{ a composite parameter};$$

$$k_1 = 1.05 - 5.7(0.4 - \rho m)^3, \text{ the factor to allow for the influence of the mechanical ratio } \rho m \text{ of longitudinal steel reinforcement};$$

$$k_2 = 1.40 - 0.10 \lambda, \text{ the factor to allow for the influence of the shear span-depth ratio } \lambda \text{ of columns.}$$

The drift ratio $\delta$ of FRP-confined columns can be estimated by Eq. (6-20) with good accuracy ($R^2=0.92$), as shown in Figure 6.25(a). Meanwhile, the Eq. (6-21) of the 5th percentile lower bound of data can also be used when a more conservative estimate of the drift ratio $\delta$ is required. It is noted that the estimation of the ratio $\delta$ by using the lower bound expression Eq. (6-21) is about 0.80 of that by the centerline expression Eq. (6-20).

In the above two expressions, the value of $k_1$ increases from 0.90 to 1.05 with the increase of $\rho m$ from 0.1 to 0.4 while the value of $k_2$ decreases from 1.1 to 0.9 with the increase of $\lambda$ from 3 to 5, which reflect the degree of the influences of these two variables. The influences of $\rho m$ and $\lambda$ on the ratio $\delta$ of FRP-confined columns can further be shown more clearly in the following example. The drift ratio $\delta$ is estimated for a FRP-confined concrete column with the composite parameter $\gamma_F = 0.2$ by using Eq. (6-20). The estimated values of $\delta$ vary between 3.4% to 4.9% due to the change of $\rho m$ and $\lambda$ within the ranges of $\rho m = 0.1$ to 0.4 and $\lambda = 3$ to 5, as shown in Table 6.2. This example indicates that the combined influences of variables $\rho m$ and $\lambda$ on the drift ratio $\delta$ of FRP-confined columns is not negligible, even though the degree of the influence by either of them is not so significant. In the absence of the detailed information about $\rho m$ or $\lambda$, the drift ratio of columns can be estimated approximately by taking the corresponding factor $k_1$ or $k_2$ as 1.0. However, the resulting error should be recognized.
Table 6.2 Predicted drift ratio of FRP-confined columns

<table>
<thead>
<tr>
<th>( \delta(%) )</th>
<th>( \rho m )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.1</td>
</tr>
<tr>
<td>( \lambda )</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>4.2</td>
</tr>
<tr>
<td>4</td>
<td>3.8</td>
</tr>
<tr>
<td>5</td>
<td>3.4</td>
</tr>
</tbody>
</table>

6.6 Strength enhancement

6.6.1 Parametric analysis

The computational results indicate that the flexural strength of cantilever concrete columns can be enhanced by the transverse confinement, especially for FRP-confined columns subjected to high levels of axial loads. This phenomenon was also reported by others for their tests on the confined concrete columns (e.g. Watson and Park, 1994; Sheikh and Yau, 2002).

In the following sections, the enhancement of flexural strength of a column is represented by the ratio of \( M_{max}/M_n \). Meanwhile, in the performance-based seismic design, the shear demand to a column is determined by the flexural strength at the column ends and the length of shear span. Thus, the enhancement of flexural strength also leads to the corresponding increase of shear demand to the column, which is represented by the ratio of \( V_{max}/V_n \) in this study. Here, \( M_{max} \) and \( V_{max} \) are the enhanced flexural strength and lateral shear demand, respectively, of a confined column computed by the program SAC, while \( M_n \) and \( V_n \) are the nominal flexural strength and shear demand of the corresponding column, respectively, same as defined previously in Chapter 4.

All the following numerical studies are based on the computational results of the circular and square columns with concrete strength up to 60 MPa, subjected to the axial load level \( k_p \) from 0.2 to 0.6. The effective data range is shown in Figure 6.26, Figure 6.27, Figure 6.29 and Figure 6.30. It has been found that the strength enhancement of steel-confined columns is closely correlated to the composite parameter \( \gamma_s = \frac{f_l}{f_c^* k_p \left( \frac{A_g}{A_c} - 1 \right)} \), while the composite parameter
\[ \gamma_F = \frac{f_l}{f_c \cdot k_p} \]

has close and constant correlation with the strength enhancement for FRP-confined columns. Thus, these two composite parameters are used in the above mentioned figures.

The flexural strength enhancement \( M_{max}/M_n \) is found to be influenced by the mechanical ratio \( \rho_m \) of the longitudinal reinforcement of columns. For examples, for a steel-confined column of \( f_c' = 30 \text{ MPa}, D = 500 \text{ mm}, k_p = 0.6 \) and \( \gamma_s = 0.67 \), the increase of \( \rho_m \) from 0.1 to 0.4 leads to an increase of the \( M_{max}/M_n \) from 1.2 to 1.4. Similarly, for an FRP-confined column of \( f_c' = 30 \text{ MPa}, D = 500 \text{ mm}, k_p = 0.6 \) and \( \gamma_F = 0.17 \), the increase of \( \rho_m \) from 0.1 to 0.4 leads to an increase of the \( M_{max}/M_n \) from 1.6 to 1.8. Thus, the influence of \( \rho_m \) is considered to predict the amount of \( M_{max}/M_n \) in the following sections.

The shear span-depth ratio \( \lambda \) of columns can also influence the increased shear demand, \( V_{max}/V_n \). For examples, for a steel-confined column with \( f_c' = 30 \text{ MPa}, D = 500 \text{ mm}, k_p = 0.6, \gamma_s = 0.9 \) and \( \rho_m = 0.3 \), the increase of \( \lambda \) from 3 to 5 leads to a decrease of the \( V_{max}/V_n \) from 1.2 to 1.1. Similarly, for an FRP-confined column with \( f_c' = 30 \text{ MPa}, D = 500 \text{ mm}, k_p = 0.6, \gamma_F = 0.17 \) and \( \rho_m = 0.3 \), the increase of \( \lambda \) from 3 to 5 leads to a decrease of \( V_{max}/V_n \) from 1.5 to 1.3. Thus, the influence of \( \lambda \) is also taken into account in the following study on \( V_{max}/V_n \).

Lastly, it is found that the effects of the cross-sectional shapes of columns and the diversity of the FRP materials on the strength enhancement are correctly accounted for if the effective confining stress \( f_l \) is estimated appropriately.

### 6.6.2 Strength enhancement of steel-confined columns

For steel-confined columns, the enhancement of flexural strength is illustrated in Figure 6.26, while Figure 6.27 displays the increase of shear demand to columns, categorized according to the different levels of axial loads \( k_p \). The centric trend line of data and its equation labelled in each chart are used to develop the expressions of \( M_{max}/M_n \) and \( V_{max}/V_n \) with the least error.
Figure 6.26  Flexural strength enhancement of steel-confined columns

(a) Data of $k_p = 0.2$

(b) Data of $k_p = 0.3$

(c) Data of $k_p = 0.4$

(d) Data of $k_p = 0.5$

(e) Data of $k_p = 0.6$
Figure 6.27 Increase of shear demand to steel-confined columns.
The following expression is proposed to predict the flexural strength enhancement of steel-confined columns:

$$\frac{M_{\text{max}}}{M_n} = 1 + 2.5k_t \cdot k_p \cdot \gamma_s$$  \hspace{1cm} (6-22)

and the increase of shear demand to columns can be estimated by:

$$\frac{V_{\text{max}}}{V_n} = 0.95 + 1.4k_t \cdot k_{ls} \cdot k_p^{2.5} \cdot \gamma_s$$  \hspace{1cm} (6-23)

where,

$$\gamma_s = \frac{f_i}{f'_c \cdot k_p \cdot \left(\frac{A_g}{A_c} - 1\right)}$$

$$k_t = 1.2\rho_m + 0.7$$, is the factor to account for the influence of the mechanical ratio \(\rho_m\) of longitudinal reinforcement in columns;

$$k_{ls} = 1.6 - 0.15\lambda$$, is the factor to account for the influence of the shear span-depth ratio \(\lambda\) of columns.

In the above two expressions, the value of \(k_t\) increases from 0.82 to 1.18 with the increase of \(\rho_m\) from 0.1 to 0.4 while the value of \(k_{ls}\) decreases from 1.15 to 0.85 with the increase of \(\lambda\) from 3 to 5, which indicate the degree of the influences of these two variables. The combined influences of variables \(\rho_m\) and \(\lambda\) can be significant as discussed in the study of the drift ratio \(\delta\) in section XYZ.

As an illustration of the procedure, a steel-confined ductile concrete column is presented as an example with the following given information: \(f'_c = 40\,\text{MPa}; \rho_m = 0.2; \lambda = 4; A_g/A_c - 1 = 0.3; \varepsilon_{su} = 0.1\) and the curvature ductility factor is targeted as \(\mu_\phi = 16\). According to Eq.(6-2b) for the steel-confined column with criterion \(\mu_\phi = 16\), the required confinement \(f_i\) is estimated under different amount of \(k_p\). Then, the enhancements of the flexural strength and shear demand of this column due to the required confinement \(f_i\) are determined by using Eqs. (6-22) and (6-23), respectively, of which the results are shown in Figure 6.28. It is seen that, when the axial load level \(k_p > 0.5\), the flexural strength and shear demand of the column are enhanced by more than 15% and 6.5%,
respectively, which is not negligible. On the other hand, the actual shear demand to this column is a little less than the nominal shear demand $V_n$ when $k_p < 0.3$ due to the $P$-$\Delta$ effect.

Recall that the Eq. (4-12) was empirically developed in Chapter 4 to predict the enhancement of flexural strength of steel-confined column, based on an experimental results in Table 4.10 for steel-confined columns under simulated seismic loads. Eq. (4-12) is reproduced below as Eq.(6-24). If factor $k_t$ is taken as unity ($\rho m = 0.25$), Eq. (6-22) is simplified to be the Eq.(6-24), which indicates that the above proposal correlates well with the experimental results.

$$\frac{M_{\text{max}}}{M_n} = 1 + 2.5k_p^2 \cdot \frac{f_I}{f_c} \cdot \left( \frac{A_g}{A_c} - 1 \right)$$  \hspace{1cm} (6-24)

---

![Graphs showing M_max/M_n and V_max/V_n vs k_p]

(a) Flexural strength enhancement 
(b) Shear demand increase

Figure 6.28  Strength enhancement of a ductile steel-confined column

### 6.6.3 Strength enhancement of FRP-confined columns

For the FRP-confined columns, the enhancement of flexural strength is illustrated in Figure 6.29, while Figure 6.30 shows the increased shear demand to columns, categorized according to the different axial load levels $k_p$. The centric trend line of data and its equation are shown in each chart, based on which the expressions of $M_{\text{max}}/M_n$ and $V_{\text{max}}/V_n$ are developed with the least error.
Figure 6.29  Flexural strength enhancement of FRP-confined columns

(a) Data of $k_p = 0.2$

(b) Data of $k_p = 0.3$

(c) Data of $k_p = 0.4$

(d) Data of $k_p = 0.5$

(e) Data of $k_p = 0.6$
Figure 6.30  Increase of shear demand to FRP-confined columns

\[ \frac{V_{\text{max}}}{V_n} = 1 + 0.14 (k_i \cdot k_{jF}) \cdot \gamma_F \]  
\[ \frac{V_{\text{max}}}{V_n} = 1 + 0.33 (k_i \cdot k_{jF}) \cdot \gamma_F \]  
\[ \frac{V_{\text{max}}}{V_n} = 1 + 0.74 (k_i \cdot k_{jF}) \cdot \gamma_F \]  
\[ \frac{V_{\text{max}}}{V_n} = 1 + 1.38 (k_i \cdot k_{jF}) \cdot \gamma_F \]  
\[ \frac{V_{\text{max}}}{V_n} = 1 + 2.20 (k_i \cdot k_{jF}) \cdot \gamma_F \]
The following regressed expression is proposed to predict the flexural strength enhancement:

\[
\frac{M_{\text{max}}}{M_n} = 1 + 14k_i \cdot k_p^{2.5} \cdot \gamma_F = 1 + 14k_i \cdot k_p^{1.5} \cdot \left(\frac{f_i}{f_c}\right)
\]  

(6-25)

and the increase of shear demand can be estimated by:

\[
\frac{V_{\text{max}}}{V_n} = 1 + 8k_{IF} \cdot k_{p}^{2.5} \cdot \gamma_F = 1 + 8k_i \cdot k_{IF} \cdot k_p^{1.5} \cdot \left(\frac{f_i}{f_c}\right)
\]  

(6-26)

where,

\[
\gamma_F = \frac{f_i}{f_c \cdot k_p}, \text{ a composite parameter};
\]

\[
k_i = 1.2 \rho_m + 0.7, \text{ is the factor to allow for the influence of the mechanical ratio } \rho_m \text{ of the longitudinal reinforcement, which is the same as for steel-confined columns;}
\]

\[
k_{IF} = 1.8 - 0.2\lambda, \text{ is the factor to account for the influence of the shear span-depth ratio } \lambda \text{ of the column.}
\]

The value of \(k_i\) increases from 0.82 to 1.18 with the increase of \(\rho_m\) from 0.1 to 0.4 while the value of \(k_{IF}\) decreases from 1.2 to 0.8 with the increase of \(\lambda\) from 3 to 5, which indicate the degree of the influences of these two variables. The combined influences of variables \(\rho_m\) and \(\lambda\) can be quite significant.

As an illustration of the use of the above proposal, a FRP-confined ductile concrete column is presented as an example with the following given information: \(f_c' = 40\ \text{MPa}; \rho_m = 0.2; \lambda = 4;\) and the curvature ductility factor is targeted at \(\mu_\phi = 13.\) According to Eq. (6-8) for the FRP-confined columns with the criterion \(\mu_\phi = 13\), the required FRP-confinement \(f_i\) is estimated under different values of \(k_p.\) Then, the enhancements of the flexural strength and shear demand of this column due to the required FRP-confinement \(f_i\) are determined by using equations (6-25) and (6-26), respectively, for which the results are shown in Figure 6.31. It is seen that, when the axial load level \(k_p > 0.4\), the flexural strength and shear demand are enhanced by more than 18% and 10%, respectively, which are not negligible.

Recall that the Eq. (4-14) was empirically developed in Chapter 4 to predict the enhancement of flexural strength for FRP-confined columns under simulated seismic loads, based on the
experimental results in Table 4.10 and is reproduced below as Eq. (6-27).

\[ \frac{M_{\text{max}}}{M_n} = 1 + 8k_p^{1.5} \cdot \left( \frac{f_{\text{l,n}}}{f_c} \right) \]  

(6-27)

Recall that the strain efficiency coefficient of FRP jackets is taken as 0.6 for large-size FRP-confined columns in the Program SAC, according to the Cui and Sheikh model (Cui and Sheikh, 2010b). Thus, the value of effective confining stress \( f_i \) in FRP-confined columns equals 0.6 of the nominal confining stress \( f_{\text{l,n}} \) in Eq. (6-27). Eq. (6-25) can be simplified to be Eq.(6-27) if the factor \( k_t = 1.0 \). This indicates that the above proposed Eq.(6-25) correlates well with the experimental results in Table 4.10.

(a) Flexural strength enhancement

(b) Shear demand increase

Figure 6.31  Strength enhancement of a ductile FRP-confined column

6.6.4 Discussion

It is evident from the preceding analyses that the strength enhancement due to the transverse confinement is relatively small only if the column is subjected to axial load level \( k_p \leq 0.3 \). However, for columns under higher levels of axial loads, the strength enhancement may be significant, as shown in Figure 6.26,Figure 6.27, Figure 6.29 andFigure 6.30. Furthermore, higher strength enhancement is generated in the FRP-confined ductile columns than in comparable steel-confined columns, because the spalling of concrete cover is effectively prevented by the external FRP-jackets.
The increase of the shear demand to confined columns is less than the corresponding enhancement of flexural strength. The examples shown in Figure 6.28 and Figure 6.31 indicate that for a confined column, the increase of shear demand is about 60% of the enhancement of flexural strength, while the remaining 40% of the flexural strength enhancement is consumed by the simultaneously enlarged $P-\Delta$ effect.

The strength enhancement of columns due to transverse confinement is neglected by most design codes currently. It is normally thought that the transverse confinement can improve the flexural ductility without remarkable enhancement of the flexural strength of columns (CAN/CSA-A23.3-04, CAN/CSA-S806-12). However, it is necessary to recognize the enhancement of flexural strength in the confined region of columns especially in the cases of FRP-retrofit, and re-evaluate the strength of adjacent members such as beam-column joints. Since the increase of flexural strength due to transverse confinement may change the internal force distribution and lead to higher strength requirement for adjacent structures which are outside the heavily confined zone. More importantly, in performance-based seismic design, the shear demand of a column is determined from its maximum probable moments and the distance between the plastic hinges at two column ends (Paulay and Priestley, 1992). The shear resistance in columns will thus be designed deficiently if the flexural strength of plastic-hinges is underestimated and the distance between the plastic hinges is overestimated, which may result in brittle shear failure of structures.

### 6.7 Case study

Two case studies are presented in the following sections in which the seismic behaviours of a steel-confined ductile column and a FRP-confined ductile column are estimated by using the expressions proposed in the preceding sections of this chapter.

First, the demand of confinement is determined according the required curvature ductility factor for each column using the expressions of the center line of $\mu_\Phi$. Based on the required transverse confinement for the column, the corresponding displacement ductility factor, lateral drift ratio and strength enhancement of each column is estimated.
6.7.1 Steel-confined column

1) Given information

This steel-confined concrete column is in a ductile moment-resisting frame and required to be designed with a curvature ductility factor of $\mu_\phi \geq 16$. The concrete strength $f'_c = 40$ MPa; axial load level $k_p = 0.5$; shear span-depth ratio $\lambda = 4$; the mechanical ratio of longitudinal reinforcement $\rho_m = 0.2$; ultimate strain of transverse steel $\varepsilon_{su} = 0.1$; and the area ratio of the concrete cover to concrete core is $A_{cov} / A_c = A_g / A_c - 1 = 0.3$.

2) Determine transverse confinement

According to Eq. (6-2b) for the steel-confined column,

$$f_l = \frac{\mu_\phi - k_2}{k_1 \cdot k_4} \cdot \left( \frac{f'_c}{1.5} \right) = \left( \frac{16 - (1)}{(1312)(1)} \right) \cdot (40)^{1.5} = 2.89 \text{ (MPa)}$$

This lateral pressure can be provided approximately by #3 spiral @ 75 mm ($f_y = 500$ MPa) in circular column of 356 mm diameter.

3) Displacement ductility factor

The displacement ductility factor of the column is then estimated by using Eq. (6-12), the centric trend line of the development of $\mu_\Delta$, as:
\[ \mu_a = k_1 \cdot \frac{f_t}{f_y^{1.5}} \left( \frac{A_g}{A_{con}} \right) + k_2 = (72.8) \frac{2.89}{(40)^{1.5}} \left( \frac{1.3}{0.3} \right) + 1.9 = 5.50 \]

where:

\[ k_1 = k_3 \cdot k_p + k_4 = (53)(0.5) + 46.3 = 72.8 \]

\[ k_2 = (\frac{6}{\lambda} + 2.3) \cdot (1 - k_p) = (\frac{6}{4} + 2.3)(1 - 0.5) = 1.9 \]

\[ k_3 = 90 \cdot (\rho_m) + 35 = (90)(0.2) + 35 = 53 \]

\[ k_4 = \frac{573}{\lambda} - 97 = \frac{573}{4} - 97 = 46.3 \]

4) Lateral drift ratio

The lateral drift ratio \( \delta \) of this steel-confined column is estimated by using Eq. (6-18) for the centric trend line as:

\[ \delta = 0.033k_1 \cdot k_2 \cdot \sqrt{\gamma_s - 0.05} = (0.033)(0.97)(1.01)\sqrt{0.482 - 0.05} = 2.12(\%) \]

Furthermore, a conservative value of \( \delta \) is estimated by using Eq. (6-19), the equation of the 5\(^{th}\) percentile lower bound of \( \delta \), as:

\[ \delta = 0.027k_1 \cdot k_2 \cdot \sqrt{\gamma_s - 0.05} = (0.027)(0.97)(1.01)\sqrt{0.482 - 0.05} = 1.74(\%) \]

where:

\[ \gamma_s = \frac{f_t}{f'_{c} \cdot k_p \cdot \left( \frac{A_g}{A_e} - 1 \right)} = \frac{2.89}{(40)(0.5)(0.3)} = 0.482 \]

\[ k_1 = 0.75 + 1.1 \rho_m = 0.75 + (1.1)(0.2) = 0.97 \]

\[ k_2 = 1.55 - 0.135 \lambda = 1.55 - (0.135)(4) = 1.01 \]

5) Strength enhancement

According to Eq. (6-22), the flexural strength enhancement of this column due to transverse steel-confinement is predicted as:
\[
\frac{M_{\text{max}}}{M_n} = 1 + 2.5k_i \cdot k_p^3 \cdot \gamma_p = 1 + (2.5)(0.94)(0.5)^3(0.482) = 1.14
\]

While the enhancement of lateral shear demand of this column is estimated by Eq. (6-23) as:

\[
\frac{V_{\text{max}}}{V_n} = 0.95 + 1.4k_i \cdot k_p \cdot k_{\lambda_s} \cdot \gamma_s = 0.95 + (1.4)(0.94)(1.0)(0.5)^2.5(0.482) = 1.06
\]

in which

\[
k_i = 1.2 \rho_m + 0.7 = (1.2)(0.2) + 0.7 = 0.94
\]

\[
k_{\lambda_s} = 1.6 - 0.15 \lambda = 1.6 - (0.15)(4) = 1.0
\]

### 6.7.2 FRP-confined column

1) Given information

This ductile column is solely confined by the transverse FRP jackets and has been designed to achieve a curvature ductility factor \( \mu_\phi \geq 13 \). The concrete strength \( f'_c = 40 \) MPa; axial load level \( k_p = 0.5 \); shear span-depth ratio \( \lambda = 4.0 \) and the mechanical ratio of longitudinal reinforcement \( \rho_m = 0.2 \).

2) Transverse confinement

According to Eq. (6-7), the curvature ductility factor \( \mu_\phi \) of FRP-confined columns can be estimated by:

\[
\mu_\phi = k_1 \cdot \left( \frac{f_i}{f_c} \right) + k_2
\]

where:

\[
k_1 = 163(k_p)^{k_3} = (163)(0.5)^{-0.136} = 179.1
\]

\[
k_2 = 1.5(k_p)^{(-0.75)} = (1.5)(0.5)^{-0.75} = 2.52
\]

\[
k_3 = 1.72 \rho_m - 0.48 \approx (1.72)(0.2) - 0.48 = -0.136
\]

Therefore, the required effective confining pressure in this column by the transverse FRP jackets is estimated as:
This lateral pressure can be provided in a 356 mm diameter column by using a 1.4 mm thick GFRP jacket with $f_{Fu} = 518$ MPa or 0.75 mm thick CFRP jacket with $f_{Fu} = 939$ MPa, if taking the strain efficiency coefficient of FRP as 0.6.

3) Displacement ductility factor

The displacement ductility factor $\mu_\Delta$ of this column is then evaluated by using Eq. (6-15), the centric trend line of the development of $\mu_\Delta$, as:

$$\mu_\Delta = k_1 \frac{f_l}{(f'_c)^{1.5}} + k_2 = \frac{535(2.34)}{(40)^{1.5}} + 2.25 = 7.20$$

where:

$$k_1 = k_3 \cdot k_p + k_4 = (246)(0.5) + 412 = 535$$

$$k_2 = \left( \frac{4}{k_p} + 3.5 \right) \cdot (1 - k_p) = \left( \frac{4}{4} + 3.5 \right) \cdot (1 - 0.5) = 2.25$$

$$k_3 = \left( \frac{1504 \times 10^3}{\lambda^5} - 322 \right) \cdot (\rho_m) + \left( 485 - \frac{480 \times 10^3}{\lambda^5} \right) = 246$$

$$k_4 = \left( \frac{410 - 962 \times 10^3}{\lambda^5} \right) \cdot (\rho_m) + \left( \frac{418 \times 10^3}{\lambda^5} + 110 \right) = 412$$

4) Lateral drift ratio

The lateral drift ratio $\delta$ of this FRP-confined column is calculated by using Eq. (6-20), the equation for the centric trend line, as:

$$\delta = 0.099k_1 \cdot k_2 \cdot \sqrt{\gamma_p} - 0.015 = (0.099)(1.004)(1.0)\sqrt{0.117 - 0.015} = 3.17(\%)$$
Furthermore, the 5th percentile lower bound of the drift ratio $\delta$ is conservatively estimated by using Eq. (6-21):

$$\delta = 0.079k_1 \cdot k_2 \cdot \sqrt{\gamma_F} - 0.015 = (0.079)(1.004)(1.0)\sqrt{0.117 - 0.015} = 2.53\%$$

where:

$$\gamma_s = \frac{f_i}{f_c \cdot k_p} = \frac{2.34}{(40)(0.5)} = 0.117$$

$$k_1 = 1.05 - 5.7(0.4 - \rho_i m)^3 = 1.05 - (5.7)(0.4 - 0.2)^3 = 1.004$$

$$k_2 = 1.40 - 0.10\lambda = 1.40 - (0.10)(4) = 1.0$$

5) Strength enhancement

According to Eq. (6-25), the flexural strength enhancement of this column due to the FRP-confinement is predicted as

$$\frac{M_{\text{max}}}{M_n} = 1 + 14k_1 \cdot k_p^{1.5} \cdot \left(\frac{f_i}{f_c}\right) = 1 + (14)(0.94)(0.5)^{1.5}\left(\frac{2.34}{40}\right) = 1.272$$

While the increase of the lateral shear demand is estimated by Eq. (6-26) as:

$$\frac{V_{\text{max}}}{V_n} = 1 + 8k_i \cdot k_{\lambda F} \cdot k_p^{1.5} \cdot \left(\frac{f_i}{f_c}\right) = 1 + (8)(0.94)(1.0)(0.5)^{1.5}\left(\frac{2.34}{40}\right) = 1.156$$

where:

$$k_i = 1.2\rho_i m + 0.7 = (1.2)(0.2) + 0.7 = 0.94$$

$$k_{\lambda F} = 1.8 - 0.2\lambda = 1.8 - (0.2)(4) = 1.0$$

6.8 Summary

Based on the computational results, the seismic behaviour of concrete columns transversely confined by steel reinforcement or the FRP wrapping is analyzed. In general, it is recognized that the behaviours of steel-confined columns and FRP-confined columns are different and should be evaluated such considering the distinct confinement mechanisms.

It is found that the seismic behaviour of confined columns is significantly influenced by the concrete strength $f_c'$, confinement level $f_i$, axial loading level $k_p$, the mechanical ratio $\rho_i m$ of
longitudinal reinforcement, the ultimate strain $\varepsilon_{su}$ of transverse steel reinforcement and the area ratio of concrete cover to concrete core. However, the cross-sectional shapes of columns has only negligible influence on the seismic behaviour of columns provided that different confinement efficiencies have been taken into account. It is also found that the seismic behaviour of FRP-confined columns is not sensitive to the diversity of FRP materials, such as CFRP or GFRP jackets.

For columns transversely confined by steel or FRP wrapping, the value of $\mu_\phi$ is influenced by the mechanical ratio of longitudinal reinforcement $\rho_{lm}$. Furthermore, shear span-depth ratio $\lambda$ can also significantly influence the displacement ductility factor $\mu_\Delta$ and the lateral drift ratio $\delta$ of columns. However, these influences on the seismic behaviour of confined concrete columns are either ignored or not correctly accounted for in many design codes.

The numerical study reveals that the flexural strength of the column can also be enhanced by transverse confinement. The amount of strength enhancement due to confinement is relatively small only if columns are subjected to the axial load level $k_p \leq 0.3$. However, for columns under higher level of axial load, the strength enhancement is significant and its influence on the adjacent structural members and the shear demand of the columns must be taken into account. It is also noted that FRP-confined columns display higher strength enhancement than the comparable steel-confined columns.

Expressions are proposed to estimate the commonly used ductility factors of concrete columns, which include the curvature ductility factor $\mu_\phi$, displacement ductility factor $\mu_\Delta$ and lateral drift ratio $\delta$. Furthermore, the flexural strength enhancement and the increase of shear demand of columns due to transverse steel or FRP-confinement are also evaluated.

Lastly, in order to demonstrate the usage of the proposed equations in this chapter, case studies of a steel-confined ductile column and a FRP-confined ductile column are presented to estimate their seismic behaviour.
Chapter 7
Design Proposals

7.1 Introduction

Design of transverse confinement in concrete columns is still a debatable topic. Seismic design provisions in various concrete codes for steel-confined column are based on diverse philosophies, criteria and procedures. For example, the seismic design provisions in ACI 318-11 for the transverse reinforcement in columns are mainly based on the philosophy that the concentric axial load capacity of a column should be maintained after the loss of concrete cover, without direct concern of ductility capacity. Meanwhile, ensuring certain levels of the curvature ductility factor of columns is the seismic design criterion in some other concrete codes, such as CAN/CSA-A23.3-04, NZC 3101:2006 and EN1998-1:2004. Figure 2.7 makes this point by highlighting the large variations in design according to different codes.

In addition to the conventional steel confinement, external FRP wrapping is becoming popular for retrofitting the seismically deficient concrete columns, as well as in new construction. But the development of design provisions is lagging behind the engineering practice in this realm. For instance, in ACI 440.2R-08 and CAN/CSA-S6-06, there is no detailed seismic design procedure for the external FRP confinement in concrete columns yet. In CAN/CSA-S806-12, two alternative methods are proposed to determine the required FRP confinement in the seismic retrofit of concrete columns, one of which is based on the criterion of curvature ductility factor $\mu_{\phi}$ of FRP-retrofitted columns while in the other the lateral drift $\delta$ is taken as the criterion of seismic performance of column.

Based on the results of numerical study in the preceding chapter and the available experimental work, design proposals are developed for the confinement of concrete columns in this chapter. The demand of transverse steel reinforcement or the external FRP jackets can be determined according to the targeted seismic performance of columns. Because various criteria are currently used in different seismic design procedures to determine the required transverse confinement of columns, the following proposals will be discussed in relation to each of these criteria individually, which include the curvature ductility factor $\mu_{\phi}$, the displacement ductility factor $\mu_{\Delta}$ and the lateral drift ratio $\delta$ of concrete columns.
7.2 Estimate of effective confinement

7.2.1 Steel-confined columns

As introduced in Chapter 2, the effective confining pressure in a steel-confined circular column is

\[ f_i = \frac{1}{2} k_e \rho_s f_{yh} \]  \hspace{2cm} (7-1)

while in a steel-confined rectangular column, the effective confining pressure in the direction under consideration is

\[ f_i = k_e \frac{A_{shy} f_{yh}}{c_y s} \]  \hspace{2cm} (7-2)

where:

- \( k_e \) = geometric coefficient of confinement effectiveness of transverse steel;
- \( \rho_s \) = volumetric ratio of steel spirals or hoops to the concrete core of a circular column, measured to the centerline of the peripheral transverse steel;
- \( f_{yh} \) = yield strength of transverse steel reinforcement;
- \( A_{shy} \) = total effective area of transverse steel in the direction under consideration within a spacing \( s \);
- \( c_y \) = width of concrete core in columns, normal to the direction under consideration and measured to the centerline of peripheral ties;
- \( s \) = center-to-center tie spacing.

The geometric coefficient of confinement effectiveness, \( k_e \), is defined as the ratio of the cross-sectional area of the effectively confined concrete core to the gross core concrete within the centerline of the peripheral reinforcement, as proposed by Sheikh and Uzumeri (1982). Because of the uneven distribution of the transverse confining stress in steel-confined columns, the value of \( k_e \) is less than 1.0 and depends mainly on the shape and configuration of the longitudinal and transverse reinforcement. To simplify the evaluation of the effective confinement in engineering practice, the following three expressions are used to get the simple yet conservative estimate of the geometric coefficient \( k_e \), suggested by Paultré and Légeron (2008).
For ductile or moderately-ductile steel-confined circular columns:

\[ k_e = 0.90 \]  

(7-3)

while for ductile rectangular columns with \( \mu_\phi \geq 16 \):

\[ k_e = 1.05 \left( \frac{A_{ch}}{A_g} \right) \frac{1}{k_n} \]  

(7-4)

and for moderately-ductile rectangular columns with \( \mu_\phi \geq 10 \):

\[ k_e = 0.95 \left( \frac{A_{ch}}{A_g} \right) \frac{1}{k_n} \]  

(7-5)

where:

- \( A_{ch} \) = cross-sectional area of concrete core, measured to the outer surface of peripheral ties;
- \( A_g \) = gross cross-sectional area of column;
- \( k_n = \frac{n_l}{n_l - 2} \), factor to allow for the influence of steel configuration in a rectangular column;
- \( n_l \) = total number of longitudinal bars in the column cross section, which are transversely supported by the corners of ties or by standard hooks of cross ties in a rectangular column.

### 7.2.2 FRP-confined columns

The fibers in the transverse FRP-confinement are assumed to be orientated in the circumferential direction of the column in this study. For circular columns transversely confined by continuous FRP jackets, the effective confinement pressure is

\[ f_t = \frac{2k_e t_F f_{Fu}}{D} \]  

(7-6)

For FRP-confined rectangular columns, it is normally required in engineering practice that the corners are rounded radius of \( r_c \geq D/15 \) before the wrapping of FRP jackets to avoid the
premature failure of FRP due to stress concentration. The side aspect ratio $h/b$ of the column is assumed to be no more than 2.0. With this condition satisfied, the geometric effectiveness of FRP confinement in a rectangular column can be taken as half of that of a circular column conservatively (Sheikh and Li, 2007), which is also supported by Eq.(2-39). Thus, the effective confinement pressure, $f_l$, of a rectangular column is estimated approximately as:

$$f_l = \frac{k_x t_F f_{Fu}}{D}$$

(7-7)

In the above two expressions, 7-6 and 7-7,

- $k_x$ = strain efficiency coefficient of FRP jackets, taken as 0.6 in this study;
- $t_F$ = total thickness of FRP jackets;
- $f_{Fu}$ = ultimate tensile strength of FRP jackets measured by standard FRP coupon tests;
- $D$ = diameter of circular column or width of a rectangular column.

If more accurate estimation of the effective FRP-confinement in columns with rectangular cross section is preferred, Eq.(2-39) proposed by ACI 440.2R-08 can be used, which was introduced in Section 2.2.2.3.

### 7.3 Curvature ductility criteria

When the curvature ductility factor $\mu_\phi$ of columns is taken as the seismic design criterion, the following design proposals are developed from the expressions of center lines of the factor $\mu_\phi$, derived previously for various kinds of columns. If more conservative design approach is required, the design proposals can be developed from the equations of the 5th percentile lower bounds following the similar procedures, which is not elaborated here.

#### 7.3.1 Steel-confined columns

For steel-confined columns with $\rho_m = 0.1$ to 0.4 and a certain target of the curvature ductility factor $\mu_\phi$, the following demand of the effective transverse confining pressure is derived from Eq. (6-3) of the center lines of $\mu_\phi$:

$$f_l = \left(\frac{\mu_\phi - k_4}{k_x \cdot k_5}\right) \cdot (f'_c)^{1.5} \cdot (k_p)^{0.5}$$

(7-8)
where:
\[ k_4 = 10 \varepsilon_{sw} \leq 1.3 \]
\[ k_5 = -1993 \rho_m + 1386 \]
\[ k_6 = 15 \rho_m - 3.5 \]

### 7.3.1.1 Steel-confined circular columns

Based on Eqs. (7-1), (7-3) and (7-8), the required volumetric ratio of spirals or hoops in the ductile circular columns with \( \mu_\phi = 16 \) is estimated as:

\[
\rho_s = 2.22 \left( \frac{16 - k_6}{k_4 \cdot k_5} \right) \cdot (k_p)^{0.5} \cdot \left( \frac{f_c'}{f_{yh}} \right)^{1.5} \quad (7-9)
\]

Similarly, for the moderately-ductile circular columns with \( \mu_\phi = 10 \), the required volumetric ratio of spirals or hoops is

\[
\rho_s = 2.22 \left( \frac{10 - k_6}{k_4 \cdot k_5} \right) \cdot (k_p)^{0.5} \cdot \left( \frac{f_c'}{f_{yh}} \right)^{1.5} \quad (7-10)
\]

The factors \( k_4, k_5 \) and \( k_6 \) in the above two expressions are the same as for Eq.(7-8).

### 7.3.1.2 Steel-confined rectangular columns

Derived from Eqs. (7-2), (7-4) and (7-8), the required effective area of the transverse steel reinforcement in the direction under consideration for ductile rectangular columns with \( \mu_\phi = 16 \) is estimated as:

\[
A_{shy} = 0.95 \cdot c_s \cdot k_n \cdot (k_p)^{0.5} \cdot \left( \frac{f_c'}{f_{yh}} \right)^{1.5} \cdot \left( \frac{16 - k_6}{k_4 \cdot k_5} \right) \cdot \left( \frac{A_g}{A_{ch}} \right) \quad (\text{mm}^2) \quad (7-11)
\]

Similarly, based on Eqs. (7-2), (7-5) and (7-8), the required effective area of the transverse reinforcement in the direction under consideration in moderately-ductile rectangular columns with \( \mu_\phi = 10 \) is
\[ A_{shy} = 1.05 \cdot c_s k_n \cdot (k_p)^{0.5} \cdot \left( \frac{f_y}{f_yh} \right)^{1.5} \cdot \left( \frac{10 - k_6}{k_4 \cdot k_5} \right) \cdot \left( \frac{A_g}{A_{ch}} \right) \text{(mm}^2) \] (7-12)

The factors \(k_4, k_5\) and \(k_6\) in the above two expressions are the same as for Eq.(7-8).

### 7.3.2 FRP-confined columns

For FRP-confined columns with a certain target of \(\mu_\phi\), the following demand of effective transverse confining pressure is derived from Eq. (6-8) of the center lines of \(\mu_\phi\) as:

\[
f_i = \left( \frac{\mu_\phi - 4.5}{k_4} \right) \cdot f'_c \cdot (k_p)^{0.5} \tag{7-13}
\]

where:

\[
k_4 = \frac{44}{(\rho, m)^{0.5}}
\]

#### 7.3.2.1 FRP-confined circular columns

Based on Eqs. (7-6) and (7-13), the required thickness of the FRP jackets for ductile circular columns with \(\mu_\phi = 13\) is estimated as:

\[
t_F = 0.161D(k_p)^{0.5} (\rho, m)^{0.5} \left( \frac{f_c'}{f_{fu}} \right) \text{(mm)} \tag{7-14}
\]

Similarly, the required thickness of the FRP jackets for moderately-ductile circular columns with \(\mu_\phi = 8\) is:

\[
t_F = 0.066D(k_p)^{0.5} (\rho, m)^{0.5} \left( \frac{f_c'}{f_{fu}} \right) \text{(mm)} \tag{7-15}
\]

#### 7.3.2.2 FRP-confined rectangular columns

Based on Eqs. (7-7) and (7-13), the required thickness of the FRP jackets for ductile rectangular columns with \(\mu_\phi = 13\) is estimated as:
Similarly, the required thickness of the FRP jackets for moderately-ductile rectangular columns with $\mu_\phi = 8$ is given by:

$$t_F = 0.322D(k_p)^{0.5}(\rho_f m)^{0.5} \left( \frac{f'_c}{f_{Fu}} \right)$$ (mm) \hspace{1cm} (7-16)$$

7.4 Displacement ductility criteria

If the displacement ductility factor $\mu_\Delta$ of columns is taken as the seismic design criterion, the following design proposals are derived for steel-confined and FRP-confined columns from the expressions of the 5th percentile lower bound of $\mu_\Delta$, regressed in the preceding chapter. Considering the data scatter of $\mu_\Delta$ observed in the study, the 5th percentile lower bounds of data are used rather than the center lines to get safer and more conservative design results.

It should be noted that the displacement ductility factor $\mu_\Delta$ may increase by about 100% with the decrease of the shear span-depth ratio $\lambda$ from 5 to 3, as discussed in Chapter 6. Obviously, the displacement ductility factor of columns cannot be estimated with reasonable accuracy without considering the influence of $\lambda$.

7.4.1 Steel-confined columns

For the steel-confined columns with a certain level of $\mu_\Delta$, the following demand of the effective confining pressure is derived from Eq. (6-14) of the 5th percentile lower bound of the factor $\mu_\Delta$:

$$f_1 = \left( \frac{\mu_\Delta - 1.1}{k_6} \right) \cdot \left( 1 - \frac{A_{ek}}{A_g} \right) \cdot (f'_c)^{1.5} \cdot (k_p)^{0.5}$$ (7-18)

where:

$$k_6 = \frac{292}{\lambda} - 30$$
7.4.1.1 Steel-confined circular columns

Based on Eqs. (7-1), (7-3) and (7-18), the required volumetric ratio of spirals or hoops in ductile circular columns with $\mu_\Delta = 4$ is estimated as:

$$\rho_s = 0.21(k_p)^{0.5} \cdot \left(\frac{f'_c}{f_{yh}}\right)^{1.5} \left(1 - \frac{A_{ch}}{A_g}\right) \cdot \left(\frac{\lambda}{9.7 - \lambda}\right)$$  \hspace{1cm} (7-19)

Similarly, for moderately-ductile circular columns with $\mu_\Delta = 2.5$, the required volumetric ratio of spirals or hoops is

$$\rho_s = 0.10(k_p)^{0.5} \cdot \left(\frac{f'_c}{f_{yh}}\right)^{1.5} \left(1 - \frac{A_{ch}}{A_g}\right) \cdot \left(\frac{\lambda}{9.7 - \lambda}\right)$$  \hspace{1cm} (7-20)

7.4.1.2 Steel-confined rectangular columns

Based on Eqs. (7-2), (7-4) and (7-18), the required area of the transverse reinforcement in the direction under consideration in ductile rectangular columns with $\mu_\Delta = 4$ is estimated as:

$$A_{shy} = 0.092 \cdot c_y sk_n (k_p)^{0.5} \cdot \left(\frac{f'_c}{f_{yh}}\right)^{1.5} \left(\frac{A_g}{A_{ch}} - 1\right) \cdot \left(\frac{\lambda}{9.7 - \lambda}\right) \quad \text{(mm}^2\text{)}$$  \hspace{1cm} (7-21)

Similarly, based on Eqs. (7-2), (7-5) and (7-18), the required area of the transverse reinforcement in the direction under consideration in moderately-ductile rectangular columns with $\mu_\Delta = 2.5$ is estimated as:

$$A_{shy} = 0.049 \cdot c_y sk_n (k_p)^{0.5} \cdot \left(\frac{f'_c}{f_{yh}}\right)^{1.5} \left(\frac{A_g}{A_{ch}} - 1\right) \cdot \left(\frac{\lambda}{9.7 - \lambda}\right) \quad \text{(mm}^2\text{)}$$  \hspace{1cm} (7-22)

7.4.2 FRP-confined columns

For FRP-confined columns with a given level of $\mu_\Delta$, the following demand of the effective confining pressure is derived from Eq. (6-17) of the 5th percentile lower bound of the factor $\mu_\Delta$:
\[ f_l = \left( \frac{\mu - 1.3}{k_b} \right) \cdot \left( f'_c \right)^{1.5} \cdot (k_p)^{0.5} \tag{7-23} \]

where: \( k_b = 730 - 100 \lambda \)

### 7.4.2.1 FRP-confined circular columns

Based on Eqs. (7-6) and (7-23), the required thickness of the FRP jackets for ductile circular columns with \( \mu = 4 \) is estimated as:

\[ t_F = \frac{0.023D}{(7.3 - \lambda)} \cdot (k_p)^{0.5} \cdot \frac{(f'_c)^{1.5}}{f_{Fu}} \tag{mm} \tag{7-24} \]

Similarly, the required thickness of the FRP jackets for moderately-ductile circular columns with \( \mu = 2.5 \) is

\[ t_F = \frac{0.01D}{(7.3 - \lambda)} \cdot (k_p)^{0.5} \cdot \frac{(f'_c)^{1.5}}{f_{Fu}} \tag{mm} \tag{7-25} \]

### 7.4.2.2 FRP-confined rectangular columns

Based on Eqs. (7-7) and (7-23), the required thickness of the FRP jackets for ductile rectangular columns with \( \mu = 4 \) is estimated as:

\[ t_F = \frac{0.045D}{(7.3 - \lambda)} \cdot (k_p)^{0.5} \cdot \frac{(f'_c)^{1.5}}{f_{Fu}} \tag{mm} \tag{7-26} \]

Similarly, the required thickness of the FRP jackets for moderately-ductile rectangular columns with \( \mu = 2.5 \) is

\[ t_F = \frac{0.02D}{(7.3 - \lambda)} \cdot (k_p)^{0.5} \cdot \frac{(f'_c)^{1.5}}{f_{Fu}} \tag{mm} \tag{7-27} \]
7.5 Lateral drift ratio criteria

The lateral drift ratio $\delta$ of columns is used as the criterion in some seismic design procedures even though the ratio $\delta$ has been recognized to just represent the lateral deformability rather than the ductility factor of columns. As an option in which the lateral drift ratio $\delta$ of columns is set as the seismic design criterion, the following design proposals are derived from the expressions of the centerlines of $\delta$ developed in Chapter 6. Of course, if more conservative design approach is desired, the design proposals can also be similarly developed from the 5th percentile lower bound. Recall that the estimated value of $\delta$ by the 5th percentile lower bound expressions is about 0.8 to 0.83 of the value according to the centerline equation.

7.5.1 Steel-confined columns

For the steel-confined columns with a certain level of targeted $\delta$, the demand for the effective confining pressure is derived from Eq. (6-18) of the centerline of the lateral drift ratio $\delta$:

$$f_i = k_p f'_c \left( \frac{A_g}{A_{ch}} - 1 \right) \left[ \left( \frac{30\delta}{k_1 \cdot k_2} \right)^2 + 0.05 \right]$$

(7-28)

where: $k_1 = 0.75 + 1.1(\rho/m)$;

$$k_2 = 1.55 - 0.135 \lambda .$$

7.5.1.1 Steel-confined circular columns

Based on Eqs. (7-1), (7-3) and (7-28), the required volumetric ratio of the spirals or hoops in the steel-confined circular columns for a given $\delta$ is estimated as:

$$\rho_s = k_p \left( \frac{f'_c}{f'_{sh}} \right) \left( \frac{A_g}{A_{ch}} - 1 \right) \left[ 2000 \left( \frac{\delta}{k_1 \cdot k_2} \right)^2 + 0.11 \right]$$

(7-29)

If the targeted deformability is taken as $\delta = 2.5\%$ for steel-confined circular ductile columns, the required volumetric ratio of the spirals or hoops is
\[ \rho_s = k_p \left( \frac{f'_c}{f_{yh}} \right) \left( \frac{A_g}{A_{ch}} - 1 \right) \left[ \frac{1.25}{(k_1 \cdot k_2)^2} + 0.11 \right] \]  

(7-30)

The factors \( k_1 \) and \( k_2 \) in the above two expressions are the same as for Eq.(7-28).

### 7.5.1.2 Steel-confined rectangular columns

Based on Eqs. (7-2), (7-4) and (7-28), the required area of the transverse reinforcement in the direction under consideration in steel-confined rectangular columns with a given \( \delta \) is estimated as:

\[ A_{sby} = (c_s \cdot sk \cdot k_p) \cdot \left( \frac{f'_c}{f_{yh}} \right) \left( \frac{A_g}{A_{ch}} - 1 \right) \left( \frac{A_g}{A_{ch}} \right) \left[ 855 \left( \frac{\delta}{k_1 \cdot k_2} \right)^2 + 0.048 \right] \]  

(mm²)  

(7-31)

If the targeted deformability of steel-confined ductile columns is taken as \( \delta = 2.5\% \), the required effective area of transverse reinforcement in the direction under consideration in steel-confined rectangular columns is:

\[ A_{sby} = (c_s \cdot sk \cdot k_p) \cdot \left( \frac{f'_c}{f_{yh}} \right) \left( \frac{A_g}{A_{ch}} - 1 \right) \left( \frac{A_g}{A_{ch}} \right) \left[ 0.534 \left( \frac{\delta}{k_1 \cdot k_2} \right)^2 + 0.048 \right] \]  

(mm²)  

(7-32)

The factors \( k_1 \) and \( k_2 \) in the above two expressions are the same as for Eq.(7-28).

### 7.5.2 FRP-confined columns

For FRP-confined columns with a certain level of the drift ratio \( \delta \), the demand of effective transverse confining pressure is derived from Eq. (6-20) of the centerline of \( \delta \):

\[ f_l = k_p f'_c \left[ \left( \frac{10.1\delta}{k_1 \cdot k_2} \right)^2 + 0.015 \right] \]  

(7-33)

where:

\[ k_1 = 1.05 - 5.7(0.4 - \rho_{ym})^3; \]

\[ k_2 = 1.4 - 0.1\lambda. \]
7.5.2.1 FRP-confined circular columns

Based on Eqs. (7-6) and (7-33), the required thickness of the FRP jackets for circular columns is estimated as:

$$ t_F = D \cdot k_p \cdot \left( \frac{f'_c}{f_{Pu}} \right) \cdot \left[ 85 \left( \frac{\delta}{k_1 \cdot k_2} \right)^2 + 0.0125 \right] \text{ (mm)} \quad (7-34) $$

Assuming that the targeted deformability of FRP-confined ductile columns is $\delta = 4\%$, the required thickness of the FRP jackets for FRP-confined circular columns is

$$ t_F = D \cdot k_p \cdot \left( \frac{f'_c}{f_{Pu}} \right) \cdot \left[ 0.136 \left( \frac{\delta}{k_1 \cdot k_2} \right)^2 + 0.0125 \right] \text{ (mm)} \quad (7-35) $$

The factors $k_1$ and $k_2$ in the above two expressions are the same as for Eq.(7-33).

7.5.2.2 FRP-confined rectangular columns

Based on Eqs. (7-7) and (7-33), the required thickness of FRP jackets for the FRP-confined rectangular columns with a certain level of the drift ratio $\delta$ is estimated as:

$$ t_F = D \cdot k_p \cdot \left( \frac{f'_c}{f_{Pu}} \right) \cdot \left[ 170 \left( \frac{\delta}{k_1 \cdot k_2} \right)^2 + 0.025 \right] \text{ (mm)} \quad (7-36) $$

Assuming that the targeted deformability of FRP-confined ductile columns is $\delta = 4\%$, the required thickness of the FRP jackets for FRP-confined ductile rectangular columns is

$$ t_F = D \cdot k_p \cdot \left( \frac{f'_c}{f_{Pu}} \right) \cdot \left[ 0.272 \left( \frac{\delta}{k_1 \cdot k_2} \right)^2 + 0.025 \right] \text{ (mm)} \quad (7-37) $$

The factors $k_1$ and $k_2$ in the above two expressions are the same as for Eq.(7-33).
7.6 Comparison of different design criteria

7.6.1 Steel-confined columns

In order to compare the confinement design using the three design criteria discussed above, a steel-confined ductile circular column is presented as an example in this section.

The properties of the circular column are as follows: concrete strength $f_{c'} = 40$ MPa; yield strength of steel spirals $f_{yh} = 500$ MPa; ultimate strain of transverse steel $\varepsilon_{su} = 0.17$; diameter of column $D = 356$ mm; $\rho_{lm} = 0.37$; clear concrete cover $cc = 20$ mm. The axial load level $k_p$ varies from 0.15 to 0.56. The column is designed for two shear span-depth ratios: $\lambda = 3$ and $\lambda = 5$. All the resistance factors are taken as unity. These properties are similar to the columns tested in this study and reported in Chapter 3.

The required volumetric ratio $\rho_s$ of the transverse steel reinforcement in this circular column is determined according to Eq. (7-9) with the seismic design criterion of $\mu_\phi = 16$, and Eq. (7-19) with the criterion $\mu_\Delta = 4$ and $\lambda = 3$ or 5, and Eq. (7-30) with the criterion $\delta = 2.5\%$ and $\lambda = 5$, respectively. The results are presented in Figure 7.1. The requirements of transverse steel for the column in a ductile moment-resisting frame using CAN/CSA-A23.3-04 and ACI 318-11 are also shown in this figure.

It is found that the design results are quite different for different design criteria. The required reinforcement for the curvature ductility-based design ($\mu_\phi = 16$) is similar to that of the drift ratio-based design ($\delta = 2.5\%, \lambda = 5$), and between the requirements of the displacement ductility-based approach ($\mu_\Delta = 4$) for columns with $\lambda = 3$ and $\lambda = 5$. The columns with a shear span-depth ratio $\lambda = 3$ require the least transverse steel while the columns with a shear span-depth ratio $\lambda = 5$ require the most transverse steel if the displacement ductility-based approach ($\mu_\Delta = 4$) is used. Among all the approaches, the drift ratio-based design ($\delta = 2.5\%, \lambda = 5$) is influenced the most by axial load level $k_p$. The steel requirement according to Clause 21.4.4.2 of CAN/CSA-A23.3-04 is very similar to the results of the drift ratio-based design. Lastly, the demand of ACI 318-11 is not influenced by the axial load level $k_p$ and its requirement falls between the two displacement ductility-based approach ($\mu_\Delta = 4$) for columns with $\lambda = 3$ and 5, respectively.
Three circular column specimens tested in this research, P27-NF-2, P40-NF-7 and P56-NF-12, are presented in the same figure together with their measured ductility factors and drift ratios for verification purpose. All the three columns achieved curvature ductility factor of $\mu_\phi > 16$. The columns had shear span-depth ratio of $\lambda = 5.2$, and achieved the displacement ductility factor about $\mu_\delta = 4$. Their drift ratio were larger than 2.5% except that the column P56-NF-12 ($k_p = 0.56$) had a drift ratio of 2.1%. The comparison shows that all the proposed design equations in this chapter lead to reasonable results for these specimens. Because some of the design equations are based on the centerlines of data, more conservative design expressions can be derived following the similar procedure by using the 5th percentile lower bound.

![Comparison of design results of a steel-confined circular column](image)

**Figure 7.1** Comparison of design results of a steel-confined circular column

### 7.6.2 FRP-confined columns

In order to demonstrate the variations in the confinement design of FRP-confined columns by using the above-mentioned three design criteria, a FRP-retrofitted ductile circular column in a ductile moment-resisting frame is presented as an example in this section.

The properties of this circular column are: concrete strength $f'_c = 40$ MPa; ultimate strength of CFRP jackets $f_{ru} = 939$ MPa; modulus of elastic of FRP jackets $E_F = 76433$ MPa; diameter of column $D = 356$ mm; $\rho_m = 0.37$. The axial load level $k_p$ varies from 0.15 to 0.56. When the
displacement ductility factor $\mu_\lambda$ is used as design criterion, the design is performed for two shear span-depth ratios of the column: $\lambda = 3$ and $\lambda = 5$. All the resistance factors are taken as unity.

The required thickness $t_F$ of the FRP-confinedment in this circular column is determined according to Eq. (7-14) with the seismic design criterion of $\mu_\phi = 13$, Eq. (7-24) with the criterion $\mu_\lambda = 4$ and $\lambda = 3$ or 5, and Eq.(7-35) with the criterion $\delta = 4\%$ and $\lambda = 5$. The results are presented in Figure 7.2. The required thicknesses of the FRP-confinement by the two alternate methods proposed in CAN/CSA-S806-12 are also shown in this figure, one of which is the curvature-based method with target $\mu_\phi = 13$ and 8 while the other is the drift-based method with target $\delta = 4\%$.

![Figure 7.2 Comparison of design results of a FRP-confined circular column](image)

It is obvious that the design results vary widely depending on the design criteria used. Almost all the methods require increased confinement with higher applied axial load levels. Among the three design approaches developed previously in this chapter, the amounts of FRP required for the curvature ductility-based and the drift ratio-based approaches are higher than the displacement-ductility-based method. The result from the displacement ductility-based approach for the column with $\lambda = 3$ shows the lowest requirement of FRP. Among all the approaches, the result of the curvature-based approach and the drift ratio-based approach are similar to the curvature-based approach in CAN/CSA-S806-12. On the other hand, the design results by using
the displacement-based approach with $\lambda = 3$ are close to those of the drift ratio-based approach in CAN/CSA-S806-12. But, it should be noted that, when using the displacement ductility-based approach for the same target of $\mu_\Delta = 4$, the required thickness of the FRP jackets to retrofit the column with $\lambda = 5$ is about twice that for the column with $\lambda = 3$.

Three FRP-confined circular columns tested in this research, P27-1CF-3, P40-1CF-8 and P56-2CF-13, together with the measured ductility factors and drift ratios during testing are presented in the same figure for verification purpose. All the specimens had more FRP-confinement than required by the proposed equations and showed much higher ductility and deformability than the corresponding design targets. The verification shows that the proposed design equations in this chapter lead to conservative and safe design results for these FRP-confined column specimens.

7.6.3 Discussion

In developing the seismic design provisions for the transverse confinement of concrete columns, the curvature ductility factor $\mu_\phi$, displacement ductility factor $\mu_\Delta$ and lateral drift ratio $\delta$ of columns are used as the performance criteria by various concrete codes and researchers.

In some of the current design codes, such as CAN/CSA-A23.3-04, NZS 3101:2006, EN1998-1:2004 and CAN/CSA-S806-12, the curvature ductility factor of concrete columns is adopted as the criterion in the seismic design provisions to determine the required confinement. It is generally thought that curvature ductility factor is the fundamental ductility parameter and can be used to evaluate the corresponding displacement ductility factor with the help of the equivalent plastic hinge length. Meanwhile, the displacement ductility factor of columns is also used as the criterion of seismic performance in literature, since it is thought that the factor $\mu_\Delta$ of individual column is the closest parameter to the global displacement ductility factor of structures, which is related with the seismic force modification factor $R_d$ based on the equal displacement rule proposed by Newmark and Veletsos (1960). The relationship between $R_d$ and the displacement ductility clearly depends on the type of structural system used. Lastly, the drift ratio of columns is adopted as criterion of seismic performance by some code provisions, such as the drift-based approach in CAN/CSA-S806-12. As explained in a commentary to this design approach (Saatcioglu, 2006), the adoption of drift ratio as the seismic design criterion of columns is based on the specifications of the drift limits during severe earthquakes in the model building codes.
such as IBC-2006 and NBCC-2005, in which the maximum allowable lateral drift ratio was limited as 2%-2.5% for most concrete frames structures.

The variations in the ductility criteria also exist in other seismic standards, such as ASCE 41-06 (2007): *Seismic Rehabilitation of Existing Buildings*. In this standard, the criteria of ductility capacity for members in steel moment-resisting frames are measured by the ratio of chord plastic rotation to its yield rotation, which equals the displacement ductility factor of the members minus one. On the other hand, the criteria of ductility capacity for members in concrete moment-resisting frames are measured by the angle of chord plastic rotation, which equals the plastic part of lateral drift ratio of the members. Obviously, the displacement ductility factor and the lateral drift are used in the same standard to define ductility capacities for members in steel and concrete frames, respectively. Currently, the above mentioned ductility criteria in ASCE 41-06 (2007) are being adopted in definitions of the default properties of plastic hinges in some widely-used design and analysis software, such as SAP2000 (CSI, 2011) and STAAD (Bentley, 2007).

Suitability of curvature ductility factor $\mu_\phi$, displacement ductility factor $\mu_\Delta$, or lateral drift ratio $\delta$ as the performance criterion in confinement design of concrete columns will depend on the type of structure, its stiffness and the overall deformation characteristics and requirements. One of these criteria may be more suitable that the other for a specific application. However, it should be realized that the required transverse confinement of columns with a certain target of ductility varies significantly if different parameters are used as the design criteria.
8.1 General

Experimental and analytical research has been conducted in this study to investigate the seismic behaviour of confined concrete columns. Based on the experimental results and the newly developed computation procedure, the relationships between transverse confinement and the seismic behaviour were discussed for reinforced concrete columns transversely confined by steel spirals or rectilinear ties or the external FRP jackets. After extensive numerical study, seismic design procedures were proposed to determine the transverse confinement required by the targeted ductile performance. Besides providing confinement to the concrete core, transverse reinforcement in columns has two other important functions: preventing the premature buckling of longitudinal reinforcement and providing sufficient shear resistance. These two functions of the transverse reinforcement are beyond the scope of this research and only the issues about confinement are studied here. However, the design of transverse reinforcement in columns should satisfy all these aspects. Conclusions of this study and recommendations for future research are presented in this chapter.

8.2 Conclusions

8.2.1 Experimental research

The experimental procedure and results were reported in detail for fifteen large-scale circular concrete columns tested under simulated seismic loads, which was the combined action of the constant axial load and lateral cyclic quasi-static load. Among these columns, eight were transversely confined by the conventional steel spirals at different spacing, while the other seven initially contained only widely spaced spirals and were then retrofitted by transverse FRP wrapping. The hysteresis loops of moment vs. curvature and shear vs. tip deflection relationships were presented and the experimental results of various ductility parameters were also reported for each specimen.

The following conclusions were drawn from this experimental research:
1. All the control specimens with only widely spaced spirals (#3@300 mm) failed in a brittle manner due to the severe premature buckling of longitudinal steel bars and crushing of concrete core. The insufficient transverse reinforcement led to very low ductility and energy dissipation capacity of columns.

2. The final failure of the columns transversely confined by closely spaced steel spirals occurred in an explosive manner, owing to the fracture of spirals close to the critical section of columns. The FRP-retrofitted columns failed due to the rupture of FRP jackets within the plastic hinge regions. It was notable that rupture of FRP jackets occurred strip by strip in a gradual manner after the FRP rupture was initiated, which was different from the explosive manner that occurred in the compression tests of FRP-confined standard concrete cylinders.

3. The seismic resistance of concrete columns can be significantly improved by appropriate transverse confinement. The test results have shown that the energy dissipation capacity and ductility levels of all the confined columns were improved because of either the closely-spaced spirals or the transverse FRP confinement.

4. The required amount of transverse confinement of columns should be increased with the increase of axial load level to achieve a certain target of ductility. But the characteristics of the ductility enhancement were different among various ductility parameters. While the measured curvature ductility factor of columns increased dramatically benefitting from the enhancement of confinement, their displacement ductility factor and lateral drift ratio could not be increased effectively beyond a certain level, mainly due to the $P$-$\Delta$ effect produced by the compressive loads acting on the structure.

5. In this experimental program, the yield strength of closely-spaced spirals could be fully utilized to provide transverse confinement in steel-confined columns, while the efficiency of the widely-spaced spirals in the control specimens as well as those in the FRP-retrofitted columns was found unreliable and had better be ignored in the confinement design.

6. The average efficiency coefficient for the FRP strain was measured to be 0.876 for the CFRP jackets and 0.796 for the GFRP jackets of the seven FRP-confined specimens in this experimental research. The coefficient was higher than most of the existing proposals mainly due to the higher quality of FRP jackets and the strain gradient when the column was subjected to
simulated seismic loads. Thus, taking into account all the available test data and recommendations from other researchers, it is safe and conservative to take the efficiency coefficient of the FRP strain at least equal to 0.6 in the design practice.

7. The equivalent length of plastic hinge on the average equaled the diameter of column section approximately and did not show significant variation with the steel configuration, level of axial load, and the type and amount of transverse reinforcement.

8. The flexural strength of concrete columns was enhanced significantly by transverse confinement, especially for the FRP-confined columns under high level of axial load. This strength enhancement is neglected in most of the current seismic design codes, which may lead to unexpected structural failure mode and deficient shear design of the columns in the performance-based seismic design.

8.2.2 Numerical study

The available experimental results are still limited for large-scale concrete columns under realistic loads due to the considerable expenses and difficulty of operation to conduct such tests in large quantities. In order to provide more information to improve the seismic design procedures, nonlinear analysis and numerical study of confined concrete columns were performed in this research, in which the monotonic pushover analysis was conducted for concrete columns transversely confined by steel spirals, ties or the FRP wrapping. The model proposed by Cui and Sheikh (2010b) for confined concrete was used in this analysis. Results from this analytical procedure were verified by extensive experimental results and simulated the seismic behaviour of confined concrete columns with reasonable accuracy.

Based on the numerical study to 13824 confined columns, which cover most of the possible ranges of variables in the engineering practice, the following conclusions are drawn:

1. Close relationship is found between the curvature ductility factor $\mu_\phi$ and the composite parameter $\alpha_s = \frac{f_l}{(f'_c)^{1.5}(k_p)^{0.5}}$ for the columns transversely confined by steel spirals or rectilinear ties. On the other hand, the composite parameter $\alpha_p = \frac{f_l}{f'_c \cdot (k_p)^{0.5}}$ can reflect the influence of $f_l,$
and $k_p$ with the least scatter for the FRP-confined columns. The curvature ductility factor $\mu_\phi$ of steel-confined and FRP-confined columns increases with the improvement of the composite parameters $\alpha_s$ and $\alpha_F$, respectively, which is consistent with the experimental observations.

2. The curvature ductility factor $\mu_\phi$ of confined columns is also influenced by the mechanical ratio $\rho_{ml}$ of longitudinal steel reinforcement. As shown by the computational results, the values of $\mu_\phi$ decrease significantly with the increase of the mechanical ratio $\rho_{ml}$ of longitudinal steel reinforcement from 0.1 to 0.4 for columns under low axial load levels, while this influence is not so significant for columns under high axial load levels.

3. For steel-confined concrete columns, the displacement ductility factor $\mu_\Delta$ has close correlation with the composite parameter $\beta_s = \frac{f_i}{(f'_c)^{1.5}(k_p)^{0.5}} \left( \frac{A_g}{A_{cov}} \right)$, i.e. the area ratio of the gross cross section to the concrete cover of the column should also be taken into account along with the parameters $f_i, f'_c$ and $k_p$. For the FRP-confined columns, the displacement ductility factor $\mu_\Delta$ has close correlation with the composite parameter $\beta_F = \frac{f_i}{(f'_c)^{1.5}(k_p)^{0.5}}$.

4. For both the steel-confined and FRP-confined columns, the displacement ductility factor $\mu_\Delta$ decreases significantly due to the increase of the shear span-depth ratio $\lambda$ of columns. In most situations, the displacement ductility factor $\mu_\Delta$ decreases to about half if the shear span-depth ratio $\lambda$ of the column increases from 3 to 5. Thus, the influence of $\lambda$ cannot be ignored in evaluating the capacity of $\mu_\Delta$ accurately. On the other hand, the mechanical ratio $\rho_{ml}$ of longitudinal steel reinforcement influences the $\mu_\Delta$ of columns to a lesser degree.

5. For the lateral drift ratio $\delta$ of steel-confined concrete columns, the composite parameter $\gamma_s = \frac{f_i}{f'_c \cdot k_p \left( \frac{A_g}{A_c} - 1 \right)}$ can reflect the influence of each individual variable with the least error.

While for the FRP-confined column the composite parameter $\gamma_F = \frac{f_i}{f'_c \cdot k_p}$ is found to have close correlation with the lateral drift ratio $\delta$. 
6. The analysis shows that the δ of both steel-confined and FRP-confined columns increases with the increase of ρm. It is also found that the δ capacity of confined columns decreases with the increase of the shear span-depth ratio λ of columns. Thus, taking into account the influence of ρm and λ improves the accuracy of the prediction of drift ratio δ.

7. The flexural strength of concrete columns is increased due to the transverse confinement, especially for columns under high level of axial loads. The enhancement of flexural strength, \( \frac{M_{\text{max}}}{M_n} \), will lead to the increase of the shear demand to columns, \( \frac{V_{\text{max}}}{V_n} \), in performance-based seismic design. It has been found that the strength enhancement of steel-confined columns is closely related to the composite parameter \( \gamma_s = \frac{f_l}{f_c' \cdot k_p \cdot \left( \frac{A_g}{A_c} - 1 \right)} \), while the composite parameter \( \gamma_p = \frac{f_l}{f_c' \cdot k_p} \) has close correlation to the strength enhancement for FRP-confined columns.

8. The flexural strength enhancement \( \frac{M_{\text{max}}}{M_n} \) of columns increases with the increase of the mechanical ratio \( \rho_m \) of the longitudinal steel reinforcement. Meanwhile, the increase of the shear span-depth ratio λ of columns leads to the decrease of \( \frac{V_{\text{max}}}{V_n} \).

8.3 Recommendations

8.3.1 Proposals for design provisions

1. Many different philosophies and procedures have been proposed or recommended for the seismic design of transverse confinement of concrete columns currently. Thus, the relevant provisions differ dramatically in various design codes. In this study, the proposals of confinement design were developed for the steel-confined and FRP-confined columns based on extensive numerical study corroborated against the test data. The simplified design expressions were derived to determine the demand of steel or FRP confinement to achieve certain levels of ductility and deformability.

2. The main parameters to describe the seismic behaviour of concrete columns include the curvature ductility factor, displacement ductility factor and lateral drift ratio, which are used by
various design codes or researchers as the seismic design criteria. It has been found that the design proposals vary dramatically if they are derived from different criteria.

3. Selection of curvature ductility factor $\mu_\phi$, displacement ductility factor $\mu_\Delta$ and lateral drift ratio $\delta$ as the performance criterion in confinement design of concrete columns will depend on the type of structure, its stiffness and the overall deformation characteristics and requirements. One of these expressions or criteria may be more suitable that the other for a specific application.

8.3.2 Further research

1. One of the reasons behind the variations in the seismic design provisions in current concrete design codes to determine the confinement of concrete columns is a lack of experimental database that is consistent and rational. A comprehensive database of the concrete columns under simulated seismic loads is important for further development of analysis and design procedures. In this database, the ranges of important variables such as the geometric properties of columns, the concrete strength, different load combinations, and different confining materials, should be studied in a consistent manner.

2. A consistent and commonly accepted design criterion needs to be established for the seismic performance of structures in the further research. Currently, the curvature ductility factor and drift ratio are used to develop the confinement provisions for columns in different design codes, while the displacement ductility factor is also used by some researchers as the criterion of the ductility capacity of columns. It indicates that the definition of ductility is mainly based on two different concepts of seismic performance: one is based on the equal-displacement rule and thus uses the ratio of the ultimate to yield deformation (such as the factors $\mu_\phi$ and $\mu_\Delta$) as the ductility criterion; the other uses the ultimate deformation capacity (such as $\delta$) as the ductility criterion. Its application to column design needs clarity.

3. A simple but commonly accepted standard protocol needs to be set up and followed in the future experimental research. It has been found that, even for the same ductility parameter, the test results reported in scientific literature also differed among various researchers due to the different standards and definitions. For instance, several different definitions of yield curvature and yield displacement are used by different researchers, so that even the same reported value of the ductility factor may represent quite different seismic performance. In addition, the transverse
component of the compressive loads was not subtracted from the resultant shear force in some experimental reports, which resulted in overestimation of the lateral load and deflection capacities. These inconsistencies lead to the difficulty of evaluating the experimental and analytical results on a common platform.

4. Further research is needed for the confined high-strength concrete (HSC) columns. With the rapid development of the high strength concrete technology, the use of HSC in seismically active zones has increased. It is already known that HSC structural elements exhibit brittle behaviour at failure, and the seismic behaviour of the HSC columns is very different from the normal strength concrete (NSC) columns. But the experimental studies on the confined HSC columns under seismic loads are still limited and inconclusive; thus, there are still many uncertainties in the seismic design of the HSC columns.

5. During major earthquakes, the axial load carrying capacity of concrete columns is the essential issue to the safety of the whole structure. Thus, a study of axial load carrying capacity of concrete columns that are subjected to large lateral excursions is needed.

6. Nonlinear analysis of concrete columns can be used to address many of the issues. More robust computation procedures should be developed to simulate the seismic behaviour of the concrete columns transversely confined by steel or the external FRP jackets or even other confinement materials. Besides providing more information for further development of design procedures, the software may also be developed as the direct design aids to determine the required confinement of concrete columns in the engineering practice.
References

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Appendices

Appendix A: Photos of specimens after test

Fifteen circular columns were tested in the experimental study, as reported in Chapters 3 and 4. The photos of specimens after test are presented in this appendix, in which the most damaged region and the failure mode are shown clearly for each column.
Figure A.1 Photos of specimen P27-NF-1 after test

(a) At the end of test

(b) Most damaged region
(a) At the end of test

(b) Most damaged region

Figure A.2 Photos of specimen P27-NF-2 after test
Figure A.3 Photos of specimen P27-1CF-3 after test

(a) At the end of test

(b) Most damaged region
(a) At the end of test

(b) Most damaged region

Figure A.4 Photos of specimen P27-2GF-4 after test
(a) At the end of test

(b) Most damaged region

Figure A.5 Photos of specimen P40-NF-5 after test
Most damages of specimen P40-NF-6 after test

Figure A.6 Photos of specimen P40-NF-6 after test
(a) At the end of test

(b) Most damaged region

Figure A.7 Photos of specimen P40-NF-7 after test
(a) At the end of test

(b) Most damaged region

Figure A.8 Photos of specimen P40-1CF-8 after test
(a) At the end of test

(b) Most damaged region

Figure A.9 Photo of specimen P40-1GF-9 after test
(a) At the end of test

(b) Most damaged region

Figure A.10 Photos of specimen P56-NF-10 after test
(a) At the end of test

(b) Most damaged region

Figure A.11 Photos of specimen P56-NF-11 after test
(a) At the end of test

(b) Most damaged region

Figure A.12 Photos of specimen P56-NF-12 after test
(a) At the end of test

(b) Most damaged region

Figure A.13 Photos of specimen P56-2CF-13 after test
(a) At the end of test

(b) Most damaged region

Figure A.14 Photos of specimen P56-3GF-14 after test
Figure A.15 Photos of specimen P56-4GF-15 after test

(a) At the end of test

(b) Most damaged region
Appendix B: Experimental curves of $P_L - \delta_L$ and $M - \Delta$ relationships of columns

For the fifteen circular columns tested in the experimental study, the hysteresis loops of the lateral shear vs. tip deflection ($V - \Delta$) relationship at the contraflexure point and the moment vs. curvature ($M - \phi$) relationship at the critical section of each column were presented in Figures 4.3 to 4.17 in Chapter 4. In this appendix, the experimental response of each specimen is also displayed graphically in the form of hysteresis loops of the applied lateral load vs. displacement ($P_L - \delta_L$) at the loading point of the MTS actuator. The hysteresis loops of the moment at the critical section vs. tip deflection ($M - \Delta$) are also presented this appendix because they are used by some researchers in the analysis of columns.
(a) Applied lateral load vs. displacement at lateral loading point ($P_L - \delta_L$) curve

(b) Moment at critical section vs. tip deflection ($M - \Delta$) Curve

Figure B.1 Experimental curves of $P_L - \delta_L$ and $M - \Delta$ relationships of P27-NF-1
(a) Applied lateral load vs. displacement at lateral loading point \((P_L - \delta_L)\) curve

(b) Moment at critical section vs. tip deflection \((M - \Delta)\) Curve

Figure B.2 Experimental curves of \(P_L - \delta_L\) and \(M - \Delta\) relationships of P27-NF-2
(a) Applied lateral load vs. displacement at lateral loading point ($P_L - \delta_L$) curve

(b) Moment at critical section vs. tip deflection ($M - \Delta$) Curve

Figure B.3 Experimental curves of $P_L - \delta_L$ and $M - \Delta$ relationships of P27-1CF-3
(a) Applied lateral load vs. displacement at lateral loading point ($P_L - \delta_L$) curve

(b) Moment at critical section vs. tip deflection ($M - \Delta$) Curve

Figure B.4 Experimental curves of $P_L - \delta_L$ and $M - \Delta$ relationships of P27-2GF-4
(a) Applied lateral load vs. displacement at lateral loading point ($P_L - \delta_L$) curve

(b) Moment at critical section vs. tip deflection ($M - \Delta$) Curve

Figure B.5 Experimental curves of $P_L - \delta_L$ and $M - \Delta$ relationships of P40-NF-5
(a) Applied lateral load vs. displacement at lateral loading point \((P_L - \delta_L)\) curve

(b) Moment at critical section vs. tip deflection \((M - \Delta)\) Curve

Figure B.6 Experimental curves of \(P_L - \delta_L\) and \(M - \Delta\) relationships of P40-NF-6
(a) Applied lateral load vs. displacement at lateral loading point ($P_L - \delta_L$) curve

(b) Moment at critical section vs. tip deflection ($M - \Delta$) Curve

Figure B.7 Experimental curves of $P_L - \delta_L$ and $M - \Delta$ relationships of P40-NF-7
(a) Applied lateral load vs. displacement at lateral loading point ($P_L - \delta_L$) curve

(b) Moment at critical section vs. tip deflection ($M - \Delta$) Curve

Figure B.8 Experimental curves of $P_L - \delta_L$ and $M - \Delta$ relationships of P40-1CF-8
(a) Applied lateral load vs. displacement at lateral loading point ($P_L - \delta_L$) curve

(b) Moment at critical section vs. tip deflection ($M - \Delta$) Curve

Figure B.9 Experimental curves of $P_L - \delta_L$ and $M - \Delta$ relationships of P40-1GF-9
Figure B.10 Experimental curves of $P_L - \delta_L$ and $M - \Delta$ relationships of P56-NF-10

(a) Applied lateral load vs. displacement at lateral loading point ($P_L - \delta_L$) curve

(b) Moment at critical section vs. tip deflection ($M - \Delta$) Curve
Figure B.11 Experimental curves of $P_L - \delta_L$ and $M - \Delta$ relationships of P56-NF-11

(a) Applied lateral load vs. displacement at lateral loading point ($P_L - \delta_L$) curve

(b) Moment at critical section vs. tip deflection ($M - \Delta$) Curve
(a) Applied lateral load vs. displacement at lateral loading point ($P_L - \delta_L$) curve

(b) Moment at critical section vs. tip deflection ($M - \Delta$) Curve

Figure B.12 Experimental curves of $P_L - \delta_L$ and $M - \Delta$ relationships of P56-NF-12
Figure B.13 Experimental curves of $P_L - \delta_L$ and $M - \Delta$ relationships of P56-2CF-13.
(a) Applied lateral load vs. displacement at lateral loading point ($P_L - \delta_L$) curve

(b) Moment at critical section vs. tip deflection ($M - \Delta$) Curve

Figure B.14 Experimental curves of $P_L - \delta_L$ and $M - \Delta$ relationships of P56-3GF-14
Figure B.15 Experimental curves of $P_L - \delta_L$ and $M - \Delta$ relationships of P56-4GF-15