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Performance of hollow core square reinforced concrete members wrapped with CFRP with different fiber orientations under static loading

Tan Duy Le
University of Wollongong, dtdl405@uowmail.edu.au

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Performance of Hollow Core Square Reinforced Concrete Members Wrapped with CFRP with Different Fiber Orientations under Static Loading

This thesis is presented as part of the requirements for the award of the Degree

MASTER OF ENGINEERING (RESEARCH)

From

UNIVERSITY OF WOLLONGONG

by

Tan Duy Le, BEng

School of Civil, Mining & Environmental Engineering

FACULTY OF ENGINEERING

2013
THESIS DECLARATION

I, Tan Duy Le, declare that this thesis, submitted in partial fulfillment of the requirements for the award of Master of Engineering (Research), in the School of Civil, Mining & Environmental Engineering, Faculty of Engineering, University of Wollongong, is wholly my own work unless otherwise referenced or acknowledged. The document has not been submitted for qualifications at any other academic institution.

Tan Duy Le

Date: ............
ABSTRACT

The present study deals with strengthening hollow core reinforced concrete (RC) columns and beams using external wrapping of Carbon fiber reinforced polymer (CFRP). The effects of fiber orientation of CFRP sheets on the axial load capacity of the columns and the shear capacity of the beams are the main parameters investigated from this study.

Hollow members have been widely used in structures due to their evident advantages when compared to the solid members, such as significantly reducing the dead load of the superstructures, leading to the decrease of foundation dimensions while maintaining the strength and stiffness of the structure. As a result, the cost of construction is reduced. However, strengthening hollow columns and beams with FRP has not been well investigated. Therefore, studying the use of FRP for strengthening hollow core columns and beams is necessary.

Sixteen specimens were designed and tested under different loading conditions in this study. These specimens were divided into two series. The first series consisting of twelve specimens and were tested as columns under concentric axial load, 25 mm and 50 mm eccentricity to investigate the effect of fiber orientation on the performance of the hollow core FRP-confined columns. The second series which included four specimens were tested as beams under four-point loading to study the improvement in shear capacity. All the specimens had square cross-section with 200 mm side dimension, 800 mm height and an 80 x 80 hole at their centers. The specimens were wrapped with CFRP sheets in different combinations of fiber orientations of 0°, 45°, and 90°.

For column specimens, test results showed that all wrapping configurations increased both the strength and ductility of hollow core square RC columns. The increase in ductility was significant for all specimens, especially for the specimens exclusively wrapped with hoop CFRP layers. Meanwhile, the increase in the compressive strength was marginal for all specimens (9% to 25% increases as compared with the control column). The fiber orientation influencing the gain in strength and ductility of the columns varies between different loading conditions. When the eccentricity
was small, the use of CFRP layers in the hoop direction was shown to be the most effective wrapping method to enhance both the strength and ductility of the columns. For columns loaded under a large eccentricity, the combinations of the hoop layers with the vertical or diagonal layers outperformed the combination of the hoop CFRP layers only in terms of strength enhancement. In contrast, the ductility of the columns with CFRP in the hoop direction was greater than that of the columns wrapped with the other combinations.

These wrapping configurations were shown to be much more efficient for the case of shear strengthening than for axial load strengthening. All wrapping configurations substantially increased the shear strength of the hollow RC beams (97% to 219% increases as compared with the control beam). The change in the fiber direction strongly affected the gain in the shear capacity and failure modes of the beam specimens. The combination of vertical CFRP layers to the longitudinal axis of the beam changed the failure mode of the beam to flexural failure, which was more ductile than shear failure. The combination of two diagonal and one vertical CFRP sheets to the longitudinal axis of the beam considerably increase the shear strength and deflection capacity of the beam.
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1 INTRODUCTION

1.1 General

Hollow core members have been used to construct many existing bridges, including cable-stayed bridges, segmental bridges, and viaducts due to their evident advantages compared to solid members. The structural advantage is significantly reduced self-weight which reduces the dead load of superstructures and hence decreases the foundation dimensions while maintaining member strength and stiffness. Further, an economic advantage is that the construction cost is significantly reduced due to the reduction of construction materials.

This thesis is concerned with the structural behavior up to failure of hollow core reinforced concrete (RC) members externally strengthened with Carbon fiber reinforced polymer sheets (CFRP).

The use of FRP for strengthening existing RC members has been recognized as a highly promising technique with many evident advantages including high strength-to-weight ratio, high corrosion and heat resistance, ease and speed of application, and practically unlimited availability in FRP sizes, geometries and dimensions. The types of FRP available for strengthening include: carbon, glass and aramid in the shapes of plates, sheets, rods and strips. The two most commonly used FRP strengthening methods are: 1) the use of externally bonded FRP plates, sheets or strips on the surface of a concrete member or 2) the use of near surface mounted FRP bars, which are embedded in concrete block via grooves. The application of FRPs to existing RC structures can be grouped into axial, shear, and flexural strengthening. External wrapping with FRP sheets for axial and shear strengthening of RC members is the focus of this thesis.

For axial strengthening of concentrically or eccentrically loaded columns using FRP external wrapping method, besides parameters such as loading conditions, cross-section shapes, size effects, compressive strength of concrete and transverse steel reinforcement ratio, fiber orientation is also an important parameter, which strongly affects the strength enhancement, ductility and failure modes of the confined columns, hence the effectiveness of the FRP. Although the effects of fiber orientation
of the FRP have been well researched, most of these studies have been carried out on solid cross-section columns. Due to the presence of a hollow part, the structural behavior of a hollow cross-section column may be different from that of a solid column. Therefore, there is still a need for studying the effects of fiber orientation of the FRP on the performance of hollow columns under different loading conditions such as concentric and eccentric loading.

Another focus of this study is the application of FRP to strengthen RC beams in shear. Shear strengthening with FRP is relatively less documented than retrofit with FRP of beams in flexure and columns through confinement due to its complexity. Among parameters that influence the gain in additional shear strength contributed by FRP, fiber orientation of the FRP is one such parameter. There has been studies conducted on this field, however, most of them has focused on solid beams. Virtually, no work has been done to date to investigate the influence of fiber orientation of the FRP on the shear performance of hollow RC beams. This is despite the fact that hollow sections have been used for many concrete members due to their structural and economical advantages, as compared with solid ones.

The wrapping orientation of the FRP on hollow core reinforced concrete members is the main focus of this study. The study deals with two major aspects. The first is the effect of fiber orientation of the FRP on the performance of hollow core square RC columns confined with CFRP under eccentric loading. The second is the effect of fiber orientation of the FRP on the shear enhancement of hollow core RC beams under pure bending loading.

1.2 Background to strengthening of RC structures

1.2.1 The need for strengthening

In the last two decades, deficiencies related to aging bridges or increase in the loading standards, have become a major concern in many countries. In the United States, more than 70% of the bridges were built before 1935 (Golabi et al. 1993), while a large proportion of the United Kingdom’s current bridge stock was built between the late 1950s and early 1970s (Flaig and Lark 2000). In the state of New South Wales, Australia, around 70% of bridges were built before 1985, with a
significant percentage in the mid 1930’s, and the peak in the 70’s (Ariyaratne et al. 2009). Due to various types of deterioration and partly because loading standards have increased over the years, many of these bridges are now defined as deficient bridges, which can be classified into two categories: functionally obsolete (bridges that no longer meet the current standards, such as narrow lanes or low-carrying capacity) or structurally deficient (bridges that require significant maintenance, rehabilitation or replacement and must be inspected every year since critical load-carrying elements were found to be in poor condition due to deterioration or damage).

In fact, American Society of Civil Engineers (2013) reported that in 2012, one in nine, or just below 11%, of the nation’s bridges were classified as structurally deficient, in which 22 states have a higher percentage of structurally deficient bridges than the national average, while five states have more than 20% of their bridges defined as structurally deficient. 24.9% of the nation's bridges were defined in either deficiency category. In the United Kingdom, it was estimated that 20% of the 155,000 road bridges had some sort of strength deficiency (William 1997). Also, Australia's infrastructure condition was assessed to be in urgent need of rehabilitation especially for the highway bridges (Engineers Australia 1999).

The current financial conditions force engineers to come up with new repair and upgrading techniques using alternative methods and materials to extend the service life of structures which may be a more economical and effective solution than building a new structure.

1.2.2 Methods for strengthening RC columns

Until the early 1990s, constructing an additional reinforced concrete cage and installing grout-injected steel jackets were the two common methods for strengthening a deficient RC column (Ballinger et al. 1993). Both methods are, however, labor intensive and sometimes difficult to implement on site. In addition to being heavy, steel jackets are also poor in resisting weather attacks (Ballinger et al. 1993, Demers and Neale 1999).

Externally bonded FRP systems have been used to strengthen and retrofit existing
concrete structures around the world since the 1990s (Sheikh et al. 2007). Due to their advanced properties as mentioned above, together with the conformity to any column shape and ability to be painted to produce the desired aesthetic design, the FRP systems have been used increasingly widely to replace steel jacketing (Ballinger et al. 1993, Saadatmanesh et al. 1996). The most common form of FRP column strengthening involves the external wrapping of FRP sheets/straps. Wrapping of FRP may also be applied to undamaged or new RC columns to increase their load carrying capacity and ductility.

Similar to steel jackets, strengthening existing RC columns using the FRP is based on a well-established fact that lateral confinement of concrete can substantially enhance its axial compressive strength and ductility. FRP confinement is accomplished by placing the fibers mainly transverse to the longitudinal axis of the column providing passive confinement, which is activated once the concrete core starts dilating as a result of Poisson’s effect and internal cracking under the axial loading. It is worthy to note that the difference between steel jacketing and FRP jacketing is that the FRP behave linearly elastic up to failure while steel behaves plastic after the yielding point and fails in a ductile manner.

Many studies have been conducted on the compressive strength and stress-strain behavior of FRP-confined concrete. However, most of these studies focused on testing solid columns and fibers in the hoop direction were used to wrap specimens. This study investigates the behavior of FRP-strengthened hollow RC columns under different axial eccentric loading. FRP sheets oriented in different directions are used to wrap specimens. The effect of fiber orientation on the performance of the columns is the main parameter of the study.

1.2.3 Methods for strengthening RC beams in shear

When a RC beam is deficient in shear, or when its shear capacity is less than the flexural capacity after flexural strengthening, shear strengthening must be considered (Teng et al. 2002).

There are several traditional methods for shear strengthening of RC beams, including: 1) locally removing concrete and replacing with new concrete after
placing extra steel stirrups; 2) cross section strengthened with steel tendons, can either be pre-stressed or not; 3) cross section strengthened with epoxy bonded steel plates, which is the forerunner for bonded CFRP plates or sheets; 4) cross section strengthened with steel straps, in which the first method was probably the most commonly used in the past, however, this method is both time consuming and in many cases not cost effective (Taljsten 2003). The above other traditional methods also have their own drawbacks. For example, in the second method, cross sectional strengthening with steel tendons, the bending reinforcement is cut off during drilling of the holes through the slab; in the third method, strengthening using epoxy steel plates, the steel plates are not anchored in the compressive zone resulting in debonding failure; and in the last method, the steel straps are quite sensitive to impact loads or vandalism (Taljsten 2003). Consequently, these strengthening methods have been replaced by the use of FRP composites for shear strengthening of RC beams due to its favorable intrinsic properties (Teng et al. 2002, Taljsten 2003). The versatility of FRPs in coping with different sectional shapes and corners is also a benefit for shear strengthening application (Teng et al. 2002).

Three types of wrapping schemes are often used to strengthen RC beams in shear, including side bonding, U-jacket and completely wrapping FRP around the section of the beams. Both FRP strips and continuous sheets have been used. FRP sheets, which are bonded transversally to the axis of the member or perpendicularly to potential shear cracks are the most commonly used methods to wrap specimens. Again, most of the studies on the application of the FRP for shear strengthening of RC beams have been carried out on solid specimens. Apparently, no study has been done on the hollow RC beams. This study is for that specific gap. The effect of fiber orientations on the shear performance of the hollow beams is another focus of the study.

1.3 Objectives

In order to investigate the effect of fiber orientations of FRP on the performance of hollow core members sixteen hollow core square RC specimens having dimensions of 200 mm x 200 mm in cross-section, 800 mm in height and having a square hole of 80 mm x 80 mm were cast and tested. Three different combinations of fiber orientations of 0°, 45°, and 90° were used to wrap confined specimens. Twelve
specimens were tested as columns under three eccentricities (0, 25, and 50 mm) for the investigations of fiber orientation effects on hollow core RC columns under eccentric loading, and four specimens were tested as beams in order to study the behavior of hollow core RC beams under pure bending loading. The main objectives of this study as follows:

- Investigate the behavior of hollow core square RC columns under axial loading;
- Investigate the effect of fiber orientation of CFRP sheets on strength, ductility and failure modes of hollow core RC columns wrapped with CFRP;
- Investigate the behavior of hollow core square RC beams under pure bending loading;
- Investigate the effect of fiber orientation on the performance of hollow core RC beams strengthened in shear with CFRP; and
- Considerations of theoretical calculations on predicting the capacity of specimens.

### 1.4 Thesis outline

The work is presented in the following chapters:

In Chapter 1, the background to strengthening RC structures is presented. The mechanism of confinement in FRP-confined concrete columns and shear strengthening of concrete beams are expressed. The reason for doing the present study is stated.

In Chapter 2, a range of studies that involve the application of CFRP composites to strengthen concrete members is reviewed. FRP in strengthening concrete columns is given in the first part of the chapter, in which the effect of fiber orientations on the performance of concrete columns is emphasized. The second part of the chapter gives the application of the FRP in strengthening concrete beams. Factors that influence the contribution of the FRP to the shear strength capacity of FRP-strengthened beams are cleared out.
In Chapter 3, existing theoretical models for predicting the strength of FRP-wrapped specimens used in this study are briefly presented. It includes the models for predicting axial load capacity of FRP-confined columns and models for predicting shear contribution due to the FRP of FRP-strengthened beams in shear.

In Chapter 4, the experimental program is described involving designing, casting, and testing specimens. The dimensions of specimens, properties of materials such as the compressive strength of concrete, strength and strain capacity of steel bars and CFRP sheets used in this study are stated. The set-up process of the test specimens into the test machine, equipment used to give the test results is also expressed.

In Chapter 5, the test results of specimens are presented and evaluated. The chapter involves analyzing behavior of columns-tested specimens and beam-tested specimens. The comparison of theoretical calculation and experimental results is also carried out.

Finally, main study findings and recommendations for future research are presented in the last chapter, Chapter 6.
2 LITERATURE REVIEW

2.1 Introduction
Externally bonded FRP systems have been used to strengthen and retrofit concrete structures for the last three decades covering most fields of strengthening such as flexural strengthening, shear strengthening, and strengthening of members subjected to axial forces or combined axial and bending forces. For flexural strengthening, bonding FRP reinforcement to the tension face of a concrete flexural member with fibers oriented along the length of the member will provide an increase in the flexural strength. For shear strengthening, FRP laminates have been bonded transversally to the axis of the member or perpendicularly to potential shear cracks. These types of bonding FRP are effective in providing additional shear strength. FRP has also been wrapped circumferentially to the face of members subjected to axial forces or combined axial and flexural forces. These hoop layers prevent the transversal dilation of the concrete core delaying the premature failure of concrete and steel reinforcements resulting in an increase in the strength and ductility of the members.

This chapter is concerned with studies on confined concrete columns under axial loading and concrete beams strengthened in shear by using externally bonded FRP systems. The effect of different fiber orientations in these studies is emphasized and discussed.

2.2 FRP in strengthening RC columns

2.2.1 Solid core columns
Most previous studies were carried out on strengthening solid RC columns in which the columns were mostly wrapped with fibers oriented in the hoop directions.

2.2.1.1 Fibers in the hoop orientation
FRP systems can be used to increase the axial compressive strength of a concrete column by providing confinement with a FRP jacket (ACI 440.2R 2008). By orienting the FRP layers transverse to the longitudinal axis of a member, the transverse or hoop fibers are similar to conventional spiral or tie reinforcing steel.
Due to its high modulus of tensile elasticity in the fiber direction, FRP layers can provide a considerable confinement pressure to the concrete core of the member under axial compressive loads. This confinement action delays the crushing of concrete, thereby increasing the compressive strength and deformation capacity of the column.

The improvement of the axial behavior of FRP confined concrete have been verified by a number of studies (Nanni and Bradford 1995; Toutanji 1999; Rochette and Labossiere 2000; Xiao and Wu 2000; Pessiki et al. 2001). Most of these studies were carried out on plain concrete cylinders, having typical dimensions of 150 mm in diameter and 300 mm in height. The specimens were wrapped with FRP layers in the hoop direction. All these studies have indicated that FRP jackets enhance the compressive strength and ductility of confined concrete. These increases substantially depend on: the properties of FRP jackets such as strain capacity and stiffness; thickness of FRP jackets such as the number of FRP plies; and types of FRP jackets such as CFRP, GFRP, and AFRP.

The effectiveness of FRP is also strongly influenced by the cross-section geometry. FRP jackets are most effective in confining members with circular cross-sections in terms of both strength and ductility. For noncircular cross-sections i.e. square and rectangular sections, the increase in the maximum axial compressive strength is marginal (Pessiki et al. 2001; Harries and Carey 2003; Rocca et al. 2007; Rocca et al. 2008).

Pessiki et al. (2001) experimentally investigated on the axial behavior of both circular and square concrete columns confined with FRP jackets. Small-scale plain concrete specimens and large-scale RC concrete specimens were prepared and tested to failure under a concentric axial load. It was concluded that cross-sectional geometry significantly influences the axial behavior of FRP jacketed specimens. The jackets provided to square cross-sectional specimens were not as efficient as those provided to specimens with circular cross sections, since square cross sections contain regions of ineffectively confined concrete. A similar conclusion was also reported by Harries and Carey (2003) when testing concrete cylindrical (305 mm
height by 152 mm diameter) and square specimens (305 mm height by 152 mm side). The shape of the specimens was observed to affect the level of confinement generated. Square specimens exhibit lower confinement levels than circular specimens having the same jacket.

Size effects have been investigated by several researchers. Rocca et al. (2007) and Rocca et al. (2008) studied on the size effect of FRP-confined RC columns. Side-aspect ratios and area-aspect ratios for non-circular specimens were investigated through testing a series of RC columns of different cross-sectional shapes (circular, square, and rectangular). Similarly, the level of confinement effectiveness for circular specimens was found to be higher than that for non-circular specimens. In terms of size effect, the level of confinement effectiveness for specimens from different cross-sectional shapes featuring the same cross-section area and similar FRP volumetric ratio decreases as the side-aspect ratio increases. Area-aspect ratio was shown not to affect the confinement effectiveness of FRP on circular specimens while no definite conclusion was drawn for non-circular specimens. Even though the increments of compressive strength in the non-circular specimens were not significant; however, they did exhibit ductility in terms of axial deformation.

These enhancements, however, are achieved only when a column is tested concentrically or when the eccentricity of the load is small. In fact, Bank (2006) has shown that the strength enhancement is only the significance for members in which compression failure is the controlling mode. When the eccentricity is large, the effectiveness of hoop FRP layers is significantly reduced because both axial action and bending action are induced. This reduction due to the effect of eccentricity is the evidence for both circular and non-circular cross-section columns (Parvin and Wang 2001; Li and Hadi 2003; Hadi 2006a, b; Maaddawy 2009; Bisby and Ranger 2010).

Li and Hadi (2003), Hadi (2006a), Hadi (2006b), Hadi (2007) and Bisby and Ranger (2010) focused on columns with circular cross-section. All the specimens in the studies by Li and Hadi (2003) and Hadi (2006a) had a diameter of 150 mm and a length of 620 mm in the test region, and were made of plain concrete. Reinforced concrete columns were also cast and served as reference specimens. All specimens
were tested under eccentricity of 42.5 mm. The only difference between the two studies is that Li and Hadi (2003) used high-strength concrete while that for Hadi (2006a) used normal-strength concrete. Results showed that external confinement with FRP could improve the performance of plain concrete columns under eccentric loading. However, the enhancement was not as significant as that of columns under concentric loading. In fact the increase in strength was marginal and only enhanced to some extent with the application of an eccentric load. In contrast, the gain in ductility was distinctive. Hadi (2006b) conducted a comparative study of eccentrically loaded FRP wrapped columns. Specimens were of 205 mm diameter and 925 mm height, and were tested under different eccentricities of 0, 25, and 50 mm. It was concluded that as expected, eccentric load reduces the maximum failure load of FRP-confined concrete columns compared to the columns under concentric load. Bisby and Ranger (2010) also reported a pronounced reduction in both strength and relative increase in strength due to confinement under flexure-dominated loading conditions. Reductions in capacity due to load eccentricity are more pronounced for FRP confined columns than for unconfined columns. Nonetheless, clear benefits of FRP wrapping, in terms of both peak load and the corresponding lateral deformation, are realized.

Parvin and Wang (2001) and Maaddawy (2009) focused their research on noncircular FRP confined columns. Both of these studies tested small-scale square concrete columns constructed with normal strength concrete. Columns tested by Parvin and Wang (2001) had no internal reinforcement and were tested under concentric and eccentric loadings. It was found that the strain gradient caused nonuniform confining pressure, which reduced the efficiency of the FRP jackets under eccentric loading. Columns tested by Maaddawy (2009) were internally reinforced. The columns were unwrapped, fully wrapped, and partially wrapped with CFRP in the hoop direction and were tested under various eccentricities with the ratio of e/h ranging from 0.3 to 0.86 (where, e is the magnitude of eccentricity, h is the depth of cross-section). The research findings indicated that the strength gain caused by FRP wrapping decreased as e/h was increased. Full FRP wrapping resulted in about 37% enhancement in compression strength at a nominal e/h of 0.3, whereas only 3% strength gain was recorded at a nominal e/h of 0.86.
The above studies showed that an eccentric load significantly affects the efficiency of FRP jacketed in the hoop direction. The level of FRP confinement increments decreases with the increase of eccentricity of the load. At a large eccentricity the strength gain in the confined columns is considerably reduced as shown in the Maaddawy’s (2009) study.

2.2.1.2 Fiber in the vertical orientation

The fact is that under eccentric compressive loading, columns buckling producing an additional bending moment (secondary moment) on the column. The increase in the applied load results in the increase of lateral deflection; hence the total eccentricity of the applied load is thereby increased. The increase in the internal moment of the column due to the increase in the total eccentricity causes the strength capacity to decrease. In addition, FRP confinement leads to greater slenderness in the column (Fitzwilliam and Bisby 2010), causing increased lateral bending acting on the column. To resist such bending moment, vertical FRP layers were provided and the experimental results were very positive.

Chaallal and Shahawy (2000) examined the effect of bidirectional FRP sheets on the performance of CFRP wrapped rectangular RC columns under combined axial-flexural loading. Full-scale specimens of 200 x 300 mm in the cross-section and 2100 mm in length in the test region were tested under eccentricities of 0, 75, 150, 300 and 400 mm. The test results indicated that the strength capacity of the confined columns was improved significantly as a result of the combined action of the longitudinal and the transverse weaves of the bidirectional composite fabric. The longitudinal fibers contributed mostly to flexural capacity, whereas the transverse ones enhanced the compressive capacity of the compression zone through confinement action. The maximum capacity gain achieved was slightly below 30% in pure compression and over 54% in pure flexure. Under combined axial force-bending moment conditions, the gain in moment capacity attained 70%. The increase in the compressive strain due to the confinement effect was also reported varying from 49 to 166%.

Hadi (2007) tested circular concrete columns externally confined with FRP, where
vertical FRP straps were added. Similar to Chaallal and Shahawy (2000), the addition of vertical CFRP straps significantly improved the performance of the columns for both strength and ductility under eccentric loading. In addition, the higher number of FRP layers in the vertical straps, the better the performance of the columns. An axial load at 50 mm eccentricity was applied to the specimens (925 mm height, 205 mm diameter). The maximum load carrying capacity of the reference column was 552 kN, while that of columns wrapped with GFRP with none, one and three vertical FRP straps were 618, 766, and 871 kN, respectively. Those columns wrapped with CFRP with none, one and three vertical FRP straps were 856, 1156, and 1240 kN, respectively. It is obvious that the higher number of FRP layers in the vertical straps gave the better performance of the wrapped columns. Additionally, the FRP wrapped columns outperformed the steel reinforced columns.

The benefit of vertical FRP sheets is more obvious with the increase in the eccentricity as shown by Hadi and Widiarsa (2012). In their study, square, reinforced concrete columns confined with CFRP were prepared and tested. The influence of the presence of vertical FRP straps was investigated. The specimens were tested under eccentricities of 0, 25, and 50 mm. The results of the study showed that the application of the vertical CFRP straps together with hoop CFRP layers significantly improved the performance of the column with large eccentricity. In the case of concentric loading, the specimens with the presence of vertical FRP straps showed an 8.4% increase in the maximum strength relative to the unwrapped specimen. Meanwhile, those of 17.8% and 14.8% increase were achieved when testing the specimens under 25 and 50 mm eccentricities, respectively. However, it can be seen that the increase in the maximum strength enhancement decreased with the increase in the eccentricity, but there was no explanation for this trend from the author.

By testing concentrically short half-scale square RC columns, Tan (2002) confirmed that increasing the amount of longitudinal fiber sheets leads to enhancement in strength and ductility of the solid, square, concentrically loaded columns. However, this increase would be possible only if they were adequately restrained from outward buckling by transverse fiber sheets.
2.2.1.3 Fiber in other orientations

Wrapping FRP in other directions has also been studied by several researchers. Rochette and Labossiere (2000) used fibers oriented at $\pm 15^\circ/0^\circ$ to wrap square concrete columns. Mirmiran and Shahawy (1997) applied $\pm 15^\circ$ fibers from the hoop direction in their concrete-filled FRP tubes. Pessiki (2001) conducted experiments on small-scale square and circular plain concrete specimens as well as large-scale square and circular reinforced concrete FRP jacketed columns under axial load. In this study, the FRP jackets were made of (a) $0^\circ/\pm 45^\circ$ multidirectional E-glass fiber reinforced polymer (GFRP) jackets E-glass fabric with 50% of its fibers oriented at $0^\circ$ angle with respect to circumferential direction, and 25% of fibers oriented at each of $\pm 45^\circ$, (b) $0^\circ$ unidirectional GFRP jackets, and (c) $0^\circ$ unidirectional carbon fiber reinforced polymer (CFRP) jackets. The compressive strength increased by 128% for small-scale circular specimens with one- ply $0^\circ/\pm 45^\circ$ GFRP jacket and 244% for circular specimens with two- ply $0^\circ$ CFRP jackets. Additionally, as compared to unjacketed specimens, axial strains at peak stress of confined specimens increased approximately seven times. Their investigation on fiber orientation was limited to a single configuration for fiber orientation at $0^\circ/\pm 45^\circ$. These specimens were not extensively analyzed in both strength and ductility aspects.

Li et al. (2006) conducted an experiment to study the effect of fiber orientation on the structural behavior of FRP-confined concrete cylinders (152.4 mm diameter, 304.8 mm height). Six fiber orientations of $0^\circ$, $90^\circ$, $\pm 45^\circ$, $30^\circ$, $60^\circ$, and their combinations were used in the study. It was found that the strength, ductility, and failure mode of the confined cylinders depend on the wrapping direction of CFRP layers and its thickness. Wrapping FRP at a certain angle in between the hoop direction and the vertical direction may result in strength lower than that of a column wrapped with FRP in the hoop direction only. Specimens wrapped with $45^\circ$ FRP showed a slight increase in the axial strain. As stated by the authors, the main reason for this slight increase in axial displacement was the insufficiency of FRP confinement ratio that was used to wrap specimen.

Sadeghian et al. (2010a) also investigated the effect of fiber orientation on concrete
cylinders. Parameters of wrap thickness, and fiber orientation were considered. The specimens were wrapped with different wrap thicknesses (1, 2, 3 and 4 layers) in different fiber orientations of 0°, 90°, ±45°, and combinations of these orientations. The test results demonstrated significant enhancement in the compressive strength, stiffness, and ductility of the CFRP-confined concrete cylinders as compared to plain cases. Longitudinal fibers have no significant effect on strength and ductility of specimens under concentric loading while the angle orientations influenced significantly the enhancement of ductility and energy dissipation of columns.

Sadeghian et al. (2010b) studied the effect of fiber orientations on rectangular RC columns strengthened with CFRP under eccentric loading. A total of seven large-scale specimens with rectangular cross section (200 mm x 300 mm) were prepared and tested under eccentric compressive loading up to failure. The overall length of specimens with two haunched heads was 2,700 mm. Different FRP thicknesses of two, three, and five layers; fiber orientations of 0°, 45°, and 90°; and two eccentricities of 200 and 300 mm were investigated. Test results and failure observation revealed that failure of all strengthened columns was governed by the rupture of longitudinal FRP layers at the tension face, which means the specimens failed in tension-controlled failure. When the strengthened columns fail in tension-controlled failure, the transverse FRP layers could not make any significant improvement on the confinement of the compression side of the section. The use of vertical FRP layers under a large eccentricity showed considerably enhancement in the maximum load capacity of strengthened columns. The failure force of S300-L4T (Specimen wrapped with 4 vertical layers and then 1 transverse layer of FRP) was equal to 354 kN, which showed a 127% increase the unstrengthened specimen (U300). The S200-L4T, which had the same wrap configuration as S300-L4T but was tested under an eccentricity of 200 mm, did not fail (more than 600 kN), while the corresponding reference specimen (U200) achieved a maximum load of about 320 kN. It is obvious that longitudinal FRP layers improved the bending stiffness and moment capacity of the specimens, but curvature capacities (the curvature was defined as the differential longitudinal strain on the top and bottom faces of the midheight section based on the plane section assumption) were not generally improved, as stated by the authors. The behavior of the specimen with inclined
orientation was a little different. In this case, not only the bending stiffness and moment capacity were enhanced, but also the curvature capacity was improved.

In the study of Parretti and Nanni (2002), tests were conducted to demonstrate the influence of fiber orientation on the performance of RC columns, in which geometry factor of cross-section was taken in account. Five circular columns and three rectangular columns were wrapped with CFRP in different fiber orientations including hoop direction and ±45° direction, and then were tested under pure axial load until failure. The test results showed that the fiber orientation can affect the behavior of RC columns, especially regarding failure mode and ductility of the specimens, and this effect is more evident for circular columns as compared with rectangular ones. Failure occurred at a small region at the mid-height of the columns wrapped with hoop CFRP laminates. Whereas, when using ±45° laminates, failure was observed to be spread over a much larger region, leading to a more ductile behavior of the strengthened columns. This enhancement of ductility performance of the column is more pronounced for circular columns than that of rectangular columns, where the effect of geometry of the cross-section on the effective of concrete core confinement is supposed to be the reason of this reduction in ductility performance. When the main goal is enhancing the load capacity, fibers in hoop direction is considered to be more effective.

The finite element analysis method has been used by several researchers to study the behavior of FRP-confined concrete columns for changes in FRP ply configuration (Parvin and Jamwal 2004, 2005; Hajsadeghi et al. 2011). The authors concluded that the rate of increase in ductility is highest for fiber oriented at an angle of 45° other than the hoop and longitudinal fiber orientations. This conclusion is noteworthy as the ductility is an important parameter of a member allowing the material to undergo large plastic deformation without failure in a brittle and abrupt manner, particularly during large earthquakes where forces are expected to exceed the yield strength of the material.

2.2.2 Hollow core columns

Recently, several studies have investigated hollow core columns. Fiber in hoop
orientation was mostly used in these studies.

Lignola et al. (2007) conducted an experiment on hollow square columns. The study aimed to evaluate the behavior of unstrengthened hollow piers and the performance of hollow piers retrofitted with FRP in terms of strength and ductility. Seven hollow square RC columns were prepared and tested. The specimens had hollow section dimensions of 360 x 360 mm, wall thickness of 60 mm, and height of 3020 mm with two haunched ends. The specimens were wrapped with two layers of CFRP in the hoop direction and tested under various eccentricities. The test results showed that horizontally oriented wraps could improve the strength and the ductility of hollow core square columns under eccentric loading. However, the strength enhancement was more pronounced for specimens loaded with a smaller eccentricity. Ductility was shown to increase with the increase of eccentricity.

Yazici and Hadi (2009) performed a study on hollow core circular concrete columns. CFRP layers in the transverse direction were also used in their experiments to wrap specimens. Eight specimens were prepared and tested under concentric, eccentric (25 and 50 mm), and pure bending loads. The specimens were 925 mm in height, 205 mm outer diameter in cross-section and 56 mm inner diameter in the hollow part. All specimens had internal steel reinforcement. Similar conclusions were drawn by the authors that the strength improvement of CFRP wrapped columns under concentric load was higher than eccentrically loaded CFRP wrapped columns. When the eccentricity was larger, the increase in strength was reduced, while that for ductility increased.

Kusumawardaningsih and Hadi (2010) studied the effectiveness of FRP confinement on hollow high strength RC columns. Both circular and square columns with either circular or square hollow core were cast and tested under axial concentric loading. All specimens had a height of 925 mm. The circular specimens had outer diameter of 205 mm, while the square specimens had side dimension of either 171 mm or 182 mm. The diameter of circular hole was 29 mm, while the side dimension of square hole was 61 mm. The FRP in the hoop direction was also used to wrap specimens in the experiment. It was found that FRP confinement increased the strength and
ductility of hollow core high strength RC columns. Hollow core columns having circular holes showed better performance as compared to columns having square holes.

2.3 FRP in strengthening RC beams

FRP can be aligned vertically, horizontally or diagonally at an angle to the beam's longitudinal axis. An angle of 45° is normally used in the case of diagonally wrapping the FRP at an angle. Literature review reveals that the fiber orientation of the FRP strongly affects the effectiveness of the FRP system in terms of shear enhancement and the propagation of inclined cracks of a FRP-strengthened beam in shear.

2.3.1 Vertical and diagonal FRP wraps

FRP orientated either perpendicularly or diagonally at 45° to the longitudinal axis of beams have been used in several studies. The findings indicated that diagonal FRPs outperformed vertical FRPs in terms of both crack propagation and shear enhancement contributed by the FRP. These studies conducted on both rectangular, and T-section beams.

Six rectangular RC beams having a 1220 mm long and a 127 mm x 203 mm cross section were cast and tested to failure in the study of Norris et al. (1997). 6 mm diameter stirrups were spaced at a distance of 8.1 in. (206 mm), greater than the effective depth $d$ so shear cracks would develop easily. These beams were simply supported and loaded close to the center to provide a region of constant shear over most of the beam, while developing small internal moments. Two fiber orientations were used in their experiment to wrap specimens according to side bonding method: two vertical CFRP layers applied perpendicularly to the beam axis; and two CFRP layers applied at $\pm45^\circ$, approximately normal and parallel to the shear cracks. It was concluded that when the CFRP fibers were placed perpendicular to cracks in the beam, a large increase in stiffness and strength was observed and a brittle failure occurred due to concrete rupture as a result of stress concentration near the ends of the CFRP. When the CFRP fibers were placed obliquely to the cracks in the beam, a smaller increase in strength and stiffness was observed; however, the mode of failure
associated with this off-axis application of CFRP was more ductile and preceded by warning signs such as snapping sounds or peeling of the CFRP.

Similar conclusion was drawn by Chaallal et al. (1998) when testing rectangular RC beams as well that diagonal side strips outperformed vertical side strips for shear strengthening in terms of crack propagation, stiffness, and shear strength. However, diagonal strips may produce premature failure as a result of concrete peel-off at strip curtailment particularly in tension stressed zones. Therefore, use of FRP U strips or U jackets could be more appropriate for extreme loading as suggested by the authors.

Diagana et al. (2003) also confirmed the better performance of the diagonal strips. The specimens in the Diagana et al. (2003) were designed with a total span of 2200 mm, and a rectangular cross-section of 130 mm width and 450 mm depth. The web reinforcement consisted of 6 mm diameter closed stirrups spaced at 300 mm center to center throughout the beam span. All specimens were tested as simple span beams subjected to a three point loading. Two fiber orientations including vertical CFRP strips and 45° CFRP strips, were used in their study similar to those used in the study of Chaallal et al. (1998). These CFRP strips were wrapped to the specimens using two wrapping methods: U-shape and complete wrapping. The test results showed that the contribution of FRP strips inclined at 45° to the ultimate force of the beam was more important than that of vertical strips. In addition, the fiber orientation of FRP strips strongly affected the effectiveness of wrapping methods. For the RC beams strengthened with vertical FRP strips, complete wrapping around the section of the beam was more efficient than the U-shape wrapping. The contribution of FRP strips in the form of complete wrapping to the ultimate force of the beam was twice more than that in the form of U-wrap. On the other hand, in the case of RC beams strengthened with diagonal strips, complete wrapping around the section of the beam was less efficient than U-shape wrapping. The contribution of FRP in the specimen completely wrapped with diagonal FRP strips was two times less than the specimen wrapped with diagonal FRP strips in U-shape. This phenomenon was due to the diagonal FRP strips of the completely wrapped beams being subjected to a twist force in the compressive region of the beams. This twist force weakened the strengthening gain effect of the FRP strips.
Square cross-sectional beams were tested by Sim et al. (2005). The beams in the study of Sim et al. (2005) had a square cross-section of 250 mm by 250 mm and effective span of 1400 mm. All specimens consist of no shear reinforcement and were tested under four-point loading configuration. Two different fiber orientations were considered, 45° and 90° to the beam axis. The orientation of the fibers was found to be an important factor. In the case of strength enhancement, the strengthening effect of 45° oriented fibers was improved by more than 10% as compared to 90° oriented fibers. In the case of crack propagation, for the specimen strengthened with 45° fiber orientation, cracks initiated at the loading area and propagated vertically to the bottom. This is not a typical diagonal shear cracking pattern and implies that the crack propagation was effectively controlled by 45° CFRP stripping strengthening. Therefore, 45° fiber orientation provided strengthened beams with greater strengthening effect and better control on the shear crack propagation as compared with 90° fiber orientation.

Jayaprakash et al. (2010) tested T cross-section beams. RC T-beams were designed with internal transverse steel reinforcement and subjected to three-point bending system. All beams were of 2980 mm span with a flange cross section of 400 mm x 100 mm and a web section of 120 mm x 240 mm. Test results showed that the shear enhancement of specimen with inclined CFRP U-strips (45°) was 16% more than the specimen with vertical CFRP U-strips (90°). In addition, during the test, it was also observed that the specimens with inclined CFRP U-strips had less crack distribution and propagation than the specimens with vertical CFRP U-strips. Thus, it shows that the proliferation of diagonal crack was significantly controlled due to the presence of the inclined discrete CFRP strip reinforcement.

2.3.2 Vertical and horizontal FRP wraps

Horizontal, vertical CFRP sheets and their combinations were used in the study of Adhikary and Mutsuyoshi (2004) to strengthen RC beams in shear. The dimensions of the tested beams were 150 mm (width) x 200 mm (depth) x 2600 mm (length). No internal shear reinforcement was provided in the desired shear failure region. All beams were tested under four-point loading over the span of 1,940 mm. Comparisons between these FRP sheet directions had been made on the effectiveness of FRP
sheets contribution to the shear strengthening. The specimen strengthened with vertical FRP sheets showed a greater ultimate strength as compared with the specimen strengthened with horizontal FRP sheets. The beam with both horizontally and vertically aligned FRP sheets showed slightly higher diagonal crack strength than the beam with only horizontally aligned FRP sheets.

2.3.3 Effects of fiber orientation in deep beams

Effects of fiber orientation have been also studied on deep beams. Several studies confirmed that fiber orientation of FRP strongly affect the shear performance of RC deep beams.

Zhang et al. (2004) used various types of CFRP and wrapping methods to strengthen RC deep beams, in which fiber orientation of the FRP was one of the studied parameters. The deep beams used in the study had a span and a cross-section of 914.4 mm, 101.6 mm, and 228.6 mm, respectively. The specimens were tested under three-point loading or four-pint loading systems in order to create different shear span ratios. The fiber orientation was found that not only affect the effectiveness of the FRP but also being influenced by other factors such as shear span-to-depth \( a/d \) ratio and wrapping method. With a determined shear span ratio, beams reinforced by 45° CFRP strips or fabrics showed to be the most efficient beams in shear strengthening as compared with beams, which were strengthened with either vertical or horizontal CFRP reinforcement. When the shear span ratio varied, the contributions of these 45°, vertical and horizontal CFRP varied. For beams with CFRP strips, as the \( a/d \) ratio decreases, the shear contribution of vertical CFRP reinforcement also decreases, while the contribution of the contribution of horizontal and 45° CFRP reinforcement increases. For beams with CFRP fabrics, as the shear \( a/d \) ratio decreases, the contribution of the U-shaped vertical CFRP laminates also decreases, while the shear contribution of the double-layered CFRP laminates increases, which includes one layer of vertical CFRP laminates and one layer of longitudinal CFRP laminates. The importance of the longitudinal CFRP reinforcement has once again been verified.

Lee et al. (2011) also found that fiber orientation of CFRP significantly influences
the shear performance of deep beams in both ultimate load and ductility. The combination of fibers in horizontal direction was found to be more pronounced than other directions. A total of 14 RC deep beams with a T-shaped cross-section were designed and manufactured in their study. Dimensions of all specimens were as follows: a wide web of 180 mm, a wide flange of 450 mm, a flange thickness of 100 mm, overall depth of 460 mm, and longitudinal length of 1800 mm. Four fiber direction combinations were used in the study to strengthen RC T-sectional deep beams in shear, including 0°/0°, 90°/90°, 90°/0°, and 45°/135°, that is horizontally/horizontally, vertically/vertically, vertically/horizontally and diagonally/diagonally to the longitudinal axis of the beams, respectively. All the FRP sheets were attached to the two sides of the beams according to the side bonding wrapping scheme. The results showed that fiber direction combinations significantly influence the contribution of CFRP to the shear performance of strengthened beams, in which the two horizontal plies of CFRP sheets (0°/0°) provide a greater contribution to both the shear strength enhancement and ductility as compared to the other fiber direction combinations. In terms of strength, 18%, 41%, 44%, and 66% increases in the ultimate load in comparison with that of the control beam were observed with the specimens strengthened with 90°/90°, 45°/135°, 90°/0°, and 0°/0° FRP plies, respectively. In terms of ductility, 97%, 115%, 140%, and 152% increases in the energy absorption capacity in comparison with that of the control beam were observed with the specimens strengthened with 90°/90°, 45°/135°, 90°/0° and 0°/0° FRP plies, respectively.

2.4 Summary

This chapter reviews a range of studies that involve the application of FRP composites to strengthen concrete members. Strengthening concrete columns and beams by externally wrapping them with FRP composites were presented in the chapter, in which effects of fiber orientation of FRP were mainly focused on discussion.

The first part of the chapter reviewed studies concerned with strengthening concrete columns. Wrapping columns with fibers in the hoop direction has been shown to be the most efficient method in improving the performance of columns under concentric
loads and small eccentric loads. When the eccentricity of the loads is large, vertical FRP sheets were introduced, and good outcomes were achieved from experimental results. The fibers oriented at an angle between 0° and 90° to the axial axis of the columns are shown to help columns undergo larger deflection for both axial and transverse deflections.

However, most of the studies were carried out on solid columns. Few studies have dealt with hollow core columns, which is the subject of the present study.

The second part of the chapter focused on studies strengthening RC beams in shear using FRP composite. For slender beams, the fibers oriented perpendicularly to the shear cracks were the most effective direction for shear strength enhancement and crack propagation control. For deep beams, the fibers oriented along the beam axis seemingly outperformed the other fiber directions. The studies in this section, however, also on solid beams. This is despite the fact that hollow sections have been used for many concrete members in practical sites due to their structural and economical advantages, as compared with solid members. Whether hollow or solid, when an RC beam is deficient in shear, or when its shear capacity is less than the flexural capacity after flexural strengthening, shear strengthening must be considered.

Existing models for predicting the strength of CFRP-strengthened specimens are presented and discussed in the next chapter.
3 BACKGROUND OF THEORITICAL CALCULATIONS

3.1 Introduction

In this chapter, existing models for predicting the strength of FRP-confined concrete columns and FRP-strengthened concrete beams in shear are presented. The reasons for selecting these models are explained. Steps of applying these models on calculating the strength of FRP-wrapped specimens are expressed in detail.

The chapter consists of two parts. The first part presents the theoretical calculations for FRP-strengthened columns and the second part presents the theoretical calculations for FRP-strengthened beams.

3.2 Theoretical considerations for strengthened columns

This part presents the calculations of the maximum FRP-confined concrete compressive strength $f'_{ccf}$ and the corresponding strain $\varepsilon'_{cc}$ based on two stress-strain models proposed by Lam and Teng (2003), and Mander et al. (1988) for confined concrete. The reason for choosing these models is the two models have been adopted by ACI Committee 440 (ACI 440.2R 2008; ACI 440.2R 2002) to predict the FRP-confined concrete strength of rectangular columns.

3.2.1 Lam and Teng’s stress-strain model for FRP-confined concrete

A stress-strain model developed by Lam and Teng (2003) for FRP-confined concrete is shown in Figure 3.1, which is based on studying circular cross-section columns. The stress-strain curve is assumed to consist of a parabolic first portion and a straight line second portion, which displays an ascending post-peak branch. The first parabolic portion meets the linear second portion smoothly, i.e. there is no change in slope between the two portions where they meet. The linear second portion ends at a point where both the compressive strength and the ultimate axial strain of confined concrete are reached. The model had taken into account the actual hoop rupture strain of the FRP instead of the ultimate material tensile strain by introducing a FRP strain efficiency factor, which is defined as the ratio of the actual FRP hoop rupture strain $\varepsilon_{h,rup}$ in FRP-confined concrete to the FRP rupture strain from flat coupon tests $\varepsilon_{frp}$. 
For non-circular cross-section columns, it is assumed that the concrete in a FRP-confined rectangular section is confined by transverse reinforcement though arching actions, and only the concrete contained by the four second-degree parabolas as shown in Figure 3.2 is fully confined while the confinement to the rest is negligible. These parabolas intersect the edges at 45°. The reduced effectiveness of an FRP jacket for a rectangular section than for a circular section has been determined by introducing shape factors $k_s_1$, $k_s_2$, which account for geometry of the cross-section. These shape factors are computed by Eqs. 3.6 and 3.7.

The maximum confined concrete compressive strength $f'_{cc}$ and the ultimate axial
strain $\varepsilon'_{cu}$ in the Lam and Teng's model (2003) are calculated by following equations:

$$\frac{f'_{cu}}{f_{co}} = 1 + k_1 k_{s1} \frac{f_i}{f_{co}}$$  \hspace{1cm} (3.1)

$$\frac{\varepsilon_{cu}}{\varepsilon_{co}} = 1.75 + k_2 k_{s2} \frac{f_i}{f_{co}} \left( \frac{\varepsilon_{h,rup}}{\varepsilon_{co}} \right)^{0.45}$$  \hspace{1cm} (3.2)

where $f'_{co}$ is the compressive strength of unconfined concrete, $\varepsilon_{co}$ is the strain at compressive strength of unconfined concrete which was taken as 0.002, recommended by the authors; the actual hoop rupture strain $\varepsilon_{h,rup}$ of FRP is computed by Eq. (3.4); $f_i$ is the maximum confinement pressure and is calculated by Eq. (3.3); $k_1, k_2$ are confinement effective coefficients, $k_1 = 3.3, k_2 = 12$;

$$f_i = \frac{2E_{frp} \varepsilon_{h,rup} t}{D}$$  \hspace{1cm} (3.3)

$$\varepsilon_{h,rup} = 0.586 \varepsilon_{frp}$$  \hspace{1cm} (3.4)

$$D = \sqrt{h^2 + b^2}$$  \hspace{1cm} (3.5)

Where, $t$ is the total thickness of FRP; $E_{frp}$ is the elastic modulus of FRP; $\varepsilon_{frp}$ is the ultimate tensile strain of FRP determined from FRP coupon test; and $D$ is the diagonal distance of the section; and $b,h$ are dimensions of the cross-section.

The two shape factors are computed as follows:

$$k_{s1} = \left( \frac{b}{h} \right)^2 \frac{A_e}{A_c}$$  \hspace{1cm} (3.6)

$$k_{s2} = \left( \frac{h}{b} \right)^{0.5} \frac{A_e}{A_c}$$  \hspace{1cm} (3.7)

where $h \geq b$, $A_e/A_c$ is the effective confinement area and is computed as follows:
In which, $R_c$ is the radius of the corners; $\rho_{sc}$ is the longitudinal steel reinforcement ratio; and $A_g$ is the gross area of cross-section.

It should be noted that Eqs. (3.1) and (3.2) for calculating the maximum confined concrete strength and the ultimate axial strain are only capable for cases where the FRP-confined concrete is said to be sufficiently confined. That is, the confinement ratio $f_l/f'_{co}$ is no less than 0.07. As such, the stress-strain curves of FRP-confined concrete ensure to show an ascending bi-linear type (Figure 3.1).

When observing the load-axial displacement response diagrams of test specimens used in this study, it is noted that all of FRP-confined specimens exhibited stress-strain curves of a descending type. In these curves, the maximum confined concrete strength $f'_{cc}$ was greater than the unconfined concrete strength $f'_{co}$. And, the axial strength of confined concrete at ultimate axial strain $f'_{cu}$ fell below $f'_{co}$. Such load-axial displacement curves are not of the shapes of the load-axial displacement curves presented in Lam and Teng (2003) (Figure 3.1).

These load-displacement curves are also not the ones of stress-strain curves proposed

\[
\frac{A_g}{A_c} = \frac{1 - \left(\frac{b}{h}\right)^2 + \left(\frac{h}{b}\right)^2}{1 - \rho_{sc} \left(\frac{3A_g}{\rho_{sc}}\right)}
\]  

(3.8)

Figure 3.3: Teng et al.’s stress-strain model for FRP-confined concrete (Teng et al. 2009)
by Teng et al. (2009). In Teng et al. (2009), when the stress-strain curves are of descending types ($\rho_K < 0.01$, where $\rho_K$ is the confinement stiffness ratio) as shown in Figure 3.3, the maximum confined concrete strength $f'_{cc}$ is assumed to be equal to the unconfined concrete strength $f'_{co}$, that is $f'_{cc}/f'_{co} = 1$. In contrast, the tested specimens showed descending load-axial displacement curves beyond $f'_{co}$.

These experimental load-axial displacement curves are, in fact, considered to be similar to the stress-strain curves originally developed by Mander et al. (1988) for steel confined concrete (Figure 3.4). The model has been shown to be applicable to FRP-confined concrete (Spoelstra and Monti 1999) and has been adopted by ACI 440.2R (2002) with suitable modifications for FRP-confined concrete. Since there has been no specific stress-strain model for FRP-confined hollow core rectangular columns at the present time, the approach presented in ACI 440.2R (2002) for solid sections was used herein to calculate compressive strength enhancement. The achieved results showed good agreement with the results obtained from the experiments.

3.2.2 Mander et al.’s stress-strain model for FRP-confined concrete

The stress-strain model developed by Mander et al. (1988) is shown in Figure 3.4. In this model, the maximum confined concrete strength $f'_{cc}$ and the corresponding strain $\varepsilon'_{cc}$ were computed by using Eqs. (3.9) and (3.10), respectively:

$$f'_{cc} = f'_{co} \left[ 2.25 \left(1 + 7.9 \frac{f_{l}}{f_{c}} - 2 \frac{f_{l}}{f_{c}} \right) \right]^{-1.25}$$

(3.9)

$$\varepsilon'_{cc} = \frac{1.71 \left(5f'_{cc} - 4f'_{co} \right)}{E_{c}}$$

(3.10)

Where, $f'_{co}$ is the unconfined concrete strength, $E_{c}$ is the modulus of elasticity of unconfined concrete. The confining pressure $f_{i}$ is given in Eq. (3.11):

$$f_{i} = \frac{\kappa_{a} \rho_{f} f_{je}}{2} = \frac{\kappa_{a} \rho_{f} \varepsilon_{je} E_{f}}{2}$$

(3.11)
Where, $\rho_f$ is a reinforcement ratio and $\kappa_a$ is a shape factor, which were computed by using Eqs. (3.12) and (3.13) for non-circular sections, respectively; $f_{fe}$ is the effective stress in FRP attained from tensile modulus of elasticity of FRP $E_f$, and effective strain in FRP $\varepsilon_{fc}$ at failure section.

The efficiency of the FRP jacket is directly related to the strain efficiency factor $\kappa_e$. This factor accounts for the difference between the actual tensile strain at rupture of FRP jacket $\varepsilon_{fc}$ and the ultimate strain reported from flat coupon tests $\varepsilon_{fu}$. Carey and Harries (2003) reported strain efficiency factor values of 0.13 and 0.16 for one medium-scale and one large-scale CFRP-wrapped square RC columns. Toutanji et al. (2010) reported values of 0.16 and 0.26 for two large-scale square RC columns. The value of 0.16 was used in the calculations of this study.

\[
\rho_f = \frac{2nt_f(b + h)}{A_{net}} \quad (3.12)
\]

\[
\kappa_a = 1 - \frac{(b - 2r)^2 + (h - 2r)^2}{3A_{net}(1 - \rho_f)} \quad (3.13)
\]

where $n$ is the number of FRP layers, $t_f$ is the thickness of one FRP layer; $r$ is the...
radius of corners of the section, $\rho_g$ is the cross sectional area ratio of the longitudinal steel reinforcement. Note that for Eqs. (3.12) and (3.13), the section gross area $A_g$ was replaced by the net area of the hollow section, i.e $A_{net} = A_g - A_{hollow}$, where $A_{hollow}$ is the area of the hollow part. Eq. (3.13) is applicable for square sections because the ratio of $b$ and $h$ equal to 1, the size effect was not considered.

3.3 Theoretical considerations for beams strengthened in shear

Three models for predicting shear capacity due to the contribution of FRP are presented in this consideration: Triantafillou and Antonopoulos (2000), Khalifa et al. (1998) and Chen and Teng (2003). These three models are chosen because, in the author's opinion, they have been widely accepted and recommended by many researchers in the literature. Of these, the first two models have been presented in the Euro code and ACI code for FRP design guidelines.

3.3.1 Triantafillou and Antonopoulos's model

Triantafillou and Antonopoulos's model (2000) is the upgrade of Triantafillou's (1998) model by developing a new effective FRP strain model to replace the one, which was proposed by Triantafillou (1998). The new effective FRP strain model is still based on regression analysis, but with a larger database of available test results, providing a specifically defined effective strain for detailed failure modes, different strengthening schemes and materials.

The shear FRP contribution according to Triantafillou and Antonopoulos (2000) is calculated by using Eq. (3.14) as follows:

$$ V_f = 0.9 \frac{\varepsilon_{f,e}}{\gamma_f} \rho_f E_f b_w d (1 + \cot \beta) \sin \beta $$

(3.14)

where, $\varepsilon_{f,e}$ is the effective FRP strain; $\gamma_f$ is the partial safety factor for FRP, taken as 1 when calculating the experimental value of FRP contribution to the shear capacity of the strengthened members; $\rho_f$ is the FRP reinforcement ratio; $E_f$ is the elastic modulus of FRP in the principal fiber orientation; $b_w$ is the minimum width of cross-section over the effective depth; $d$ is the effective depth of cross-section; and $\beta$ is the
angle between the principal fiber orientation and longitudinal axis of the member.

The effective strain of FRP $\varepsilon_{\text{f,e}}$ is computed by using Eq. (3.15), (3.16), or (3.17) depending on strengthening schemes and FRP materials.

Fully wrapped CFRP:

$$\varepsilon_{\text{f,e}} = 0.17 \left( \frac{f_c^{2/3}}{E_f \rho_f} \right)^{0.30} \varepsilon_{\text{f,u}}$$

(3.15)

Side or U-shaped CFRP jackets:

$$\varepsilon_{\text{f,e}} = \min \left[ 0.65 \left( \frac{f_c^{2/3}}{E_f \rho_f} \right)^{0.56}, 0.17 \left( \frac{f_c^{2/3}}{E_f \rho_f} \right)^{0.30} \varepsilon_{\text{f,u}} \right]$$

(3.16)

Fully wrapped with AFRP:

$$\varepsilon_{\text{f,e}} = 0.048 \left( \frac{f_c^{2/3}}{E_f \rho_f} \right)^{0.47} \varepsilon_{\text{f,u}}$$

(3.17)

where, $f_c$ is the compressive strength of unconfined concrete; $\varepsilon_{\text{f,u}}$ is the ultimate tensile strength of FRP obtained from testing FRP flat coupons.

3.3.2 Khalifa et al.’s model

Khalifa et al. (1998) also proposed a modification to Triantafillou’s (1998) effective strain model. The authors used the ratio of effective stress (or strain) in the FRP to its ultimate strength (or strain), $R$, which is given by $R = \frac{\varepsilon_{\text{f,e}}}{\varepsilon_{\text{f,rup}}} = \frac{f_c}{f_{\text{frp}}}$. Regression of experimental data led to the following expression for the ratio of effective stress $R$:

$$R = 0.5622 \left( \rho_{\text{FRP}} E_{\text{FRP}} \right)^2 - 1.2188 \rho_{\text{FRP}} E_{\text{FRP}} + 0.778 \leq 0.50$$

(3.18)

The shear strength due to the FRP is then computed as follows:
where \( d_f \) is the effective depth of the FRP is the distance from the tension steel centroid to the bottom of the top flange; \( s_f \) is the center-to-center spacing of FRP strips.

The Eq. (3.19) is valid only for CFRP continuous sheets or strips and is suitable if the failure mechanism is controlled by FRP sheets rupture.

### 3.3.3 Chen and Teng's model

An extensive work performed by Chen and Teng (2003) resulted in one of the most widely used shear models. A reduction factor for the stress is also used in their work. The equation for calculating the FRP contribution to shear capacity is presented as following form:

\[
V_{\text{FRP}} = \frac{A_f f_{f,e} \left( \sin \beta + \cos \beta \right) d_f}{s_f}
\]

(3.19)

where, \( t_f \) is the thickness of FRP; \( w_f \) is the width of the FRP strips (perpendicular to the fiber orientation); \( \beta \) is the angle of fibers in the FRP measured from the longitudinal axis of the beam; \( \theta \) is the inclination angle of the critical shear crack with respect to the longitudinal axis; \( h_{f,e} \) is the effective height of the FRP on the sides of the beam, for fully wrapping \( h_{f,e} = 0.9d \), where \( d \) is the effective depth of the beam.

\( f_{f,e} \) is the effective stress in the FRP intersected by a shear crack at the ultimate state which can be written as in Eq. (3.21):

\[
f_{f,e} = D_f f_f
\]

(3.21)

where, \( f_f \) is the tensile strength of the FRP which obtained from the elasticity modulus of the FRP \( E_f \) and ultimate tensile strain of FRP from FRP coupon test \( \varepsilon_{f,u} \); \( D_f \) is the stress distribution factor. For fully wrapping FRP, \( D_f \) is taken as 0.5.
3.4 Summary

The chapter presents existing models to predict strength of test specimens in this experimental program. In case of predicting the strength of FRP-wrapped columns, the two models developed for confined concrete of Lam and Teng (2003b) and Mander et al. (1988) were chosen to discuss. The two models have been adopted by ACI Committee 2R.440 (2008 and 2002) for FRP-confined concrete. The model originally developed by Mander et al. (1988), which is presented in ACI 2R.440 (2002) was selected to calculate the strength of column-tested specimens in this experimental program. The reason is that all of the column-tested specimens showed axial load-displacement curves with a descending type, which is similar to stress-strain model presented by Mander et al. (1988). While Lam and Teng (2003b) limits the application of their model for cases that show stress-strain curves with an ascending type.

In the case of beam-tested specimens, three models proposed by Triantafillou and Antonopoulos (2000), Khalifa et al. (1998), and Chen and Teng (2003) were selected to calculate the additional shear strength due to the FRP of CFRP-strengthened beams used in this experiment. The calculation steps using these models have been presented. The wide acceptance of these models in the literature and the adoption of ACI and Euro codes are the reason for them to be chosen.

The comparison between the theoretical calculation results using these models and those from the experiment are discussed and presented in the next chapter.
4 EXPERIMENTAL PROGRAM

4.1 Introduction

A total of sixteen hollow core square RC columns were designed, and tested. The combinations of three different fiber orientations of 0°, 45° and 90° were used to wrap the specimens. Twelve specimens were tested as columns under different eccentricities in order to investigate the effect of fiber orientations on the behavior of hollow core RC columns under eccentric loading, and four specimens were tested as beams to study shear performance of hollow core RC beams. All tests were carried out at the laboratories of the School of Civil, Mining, and Environmental Engineering, University of Wollongong.

4.2 Specimen design

All specimens were made of reinforced concrete with the same amount of internal steel reinforcement and were designed according to the requirements of the Australian Standard AS 3600 (2009).

![Figure 4.1: Details of test specimens and reinforcement](image)

Figure 4.1: Details of test specimens and reinforcement
The specimens had a square section with 200 mm side dimension, 800 mm height, and an 80x80 mm hole at their center. All corners of the specimens were rounded to 32 mm radius to prevent premature failure of FRP wraps due to stress concentration. The specimen dimensions were chosen based on the available equipment. The specimens were considered to be short columns. 60 mm wall thickness was designed to ensure a clear concrete cover of 20 mm was maintained at both outer and hole faces of the column as specified in AS 3600 (2009). With the wall width-to-thickness ratio of 1.33, which is less than 15, the failure mode of the compression flange will be controlled by crushing of the concrete instead of local buckling (Taylor et al. 1995). The size effect, however, is not considered in this study. Two types of steel reinforcement were used. 12 mm diameter deformed bars N12 (500 MPa nominal tensile strength) were used for longitudinal reinforcement. 6 mm diameter plain bars R6 (250 MPa nominal tensile strength) were used for transverse reinforcement and were placed at 100 mm spacing. A steel cage is shown in Figure 4.2.

Figure 4.2: Steel cage of the specimens
For all columns, four strain gages were bonded at mid-height of the four longitudinal steel bars, and two strain gages were bonded on the tie at mid-height of the column. Two types of strain gages were used: PFL-10-11-1LJB (10 mm length, 2% limit strain) was used for longitudinal steel bars, and FLA-11-1L (5 mm length, 1% limit strain) for ties. The strain gages were supplied by Bestech Australia PTY LTD Company, Dingley, Vic 3172, Australia. Details of arrangement of strain gages are given in Figure 4.3. These strain gages were used to check the strain in the steel reinforcements.

![Diagram showing the arrangement of strain gages](image)

**Figure 4.3**: Location of strain gages at mid-height of the column

### 4.3 Casting specimens

A wooden formwork having sixteen square holes was used to cast the specimens (Figure 4.4). To create the inside hole, sixteen 80 mm by 80 mm wooden boxes were also made and placed at the center of each mold. Four foam arches with 32 mm radius were placed at four corners of each hole to produce round corners for the specimens. 40 MPa nominal compressive strength concrete was used in this experimental program and was supplied by a local supplier in one batch of concrete. The concrete had 130 mm slump ensuring its workability. The concrete was then placed into the formwork at three stages. At each stage, vibration was carried out using two vibrators to ensure compaction of concrete. 100 mm diameter and 200 mm height cylinders were also cast to determine the properties of concrete (Figure 4.5).
After casting, the specimens were cured in their forms in moist conditions. Wet hessian rugs were placed on the top of all the specimens and were watered twice a day to keep them moist. The formwork was removed after 21 days of casting the specimens.

**Figure 4.4:** A wooden formwork

**Figure 4.5:** Pouring concrete: specimens and cylinders
4.4 Wrapping FRP

Before wrapping FRP, the concrete surface was prepared carefully. The concrete surface of all specimens was found to be intact after removing the formwork. Therefore, there was no need for repair or patch, except the surface at corners due to using foam. These locations were then ground using a grinder and a steel brush to make them smooth (Figure 4.6).

![Preparing surface](image1)

**Figure 4.6:** Preparing surface

![Finish surface preparation](image2)

**Figure 4.7:** Finish surface preparation
The sixteen specimens were sub-divided into four groups with four specimens each (Figure 4.7). The specimens of the first group (RC Group) without any CFRP wraps served as reference specimens. The specimens of the second group (HF Group) were laterally wrapped with three-layer CFRP with respect to the specimen’s axial axis. The specimens of the third group (VHF Group) were firstly wrapped with one CFRP layer along the specimen’s axial axis, and then laterally wrapped with two CFRP layers. All the specimens of the last group (AHF Group) were wrapped with two CFRP layers oriented at ±45° with respect to specimen’s axial axis, and then laterally wrapped with one layer of CFRP (Table 4.1).

Unidirectional CFRP fiber sheets (CARBON-UNI340GM-75MM) were used in this experimental program to wrap the specimens using wet layup system (Figure 4.8). The nominal width and thickness of the CFRP sheet were 75 mm and 0.45 mm, respectively. The properties of FRP are presented in Section 4.5.3. The adhesive was mixed from epoxy resin and slow hardener at a ratio of 5:1 as recommended by the manufacturer.

**Figure 4.8:** FRP system

The wrapping procedure was done as follows (Figures 4.10, 4.11, 4.12, and 4.13). The surface of the specimens was coated with a thin layer of epoxy resin first, and then the first layer of CFRP was applied at the design orientation. The first layer of FRP was then coated with epoxy again before the application of the second layer of FRP. The process was repeated until all the design FRP layers were applied. An overlap of 100 mm was made in the last revolution and was applied only for the layers in the hoop direction. Extra two-layer lateral CFRP and vertical FRP straps of
75 mm length were applied at the ends of all specimens in order to protect the two ends against premature failure due to stress concentration and from early cracking on the tension sides of the ends under eccentric loading (Refer to figures sequentially 4.1 to 4.13 in the previous pages).

**Table 4.1: CFRP wrapping configuration**

<table>
<thead>
<tr>
<th>Group (4 specimen each)</th>
<th>Dimension (mm)</th>
<th>Internal steel reinforcement.</th>
<th>CFRP configuration</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC</td>
<td>800 mm length; 200 x 200 mm cross section; 80 x 80 mm square hollow inside</td>
<td>4N12 for longitudinal steel reinforcement; R6@100 CTS for transverse steel reinforcement</td>
<td>None</td>
</tr>
<tr>
<td>HF</td>
<td>800 mm length; 200 x 200 mm cross section; 80 x 80 mm square hollow inside</td>
<td>4N12 for longitudinal steel reinforcement; R6@100 CTS for transverse steel reinforcement</td>
<td>Three hoop layers to the columns’ axial axis</td>
</tr>
<tr>
<td>VHF</td>
<td>800 mm length; 200 x 200 mm cross section; 80 x 80 mm square hollow inside</td>
<td>4N12 for longitudinal steel reinforcement; R6@100 CTS for transverse steel reinforcement</td>
<td>One vertical and two hoop layers to the columns’ axial axis</td>
</tr>
<tr>
<td>AHF</td>
<td>800 mm length; 200 x 200 mm cross section; 80 x 80 mm square hollow inside</td>
<td>4N12 for longitudinal steel reinforcement; R6@100 CTS for transverse steel reinforcement</td>
<td>Two 45°-oriented layers and one hoop layers to the columns’ axial axis</td>
</tr>
</tbody>
</table>

After completing wrapping, all specimens were left for at least 21 days for FRP curing.

**Figure 4.9**: Applying extra 75 mm vertical and lateral FRP layers
Figure 4.10: Group RC (Reference specimens) with extra vertical and transverse layers at the ends.

Figure 4.11: Steps of wrapping specimens in Group HF (wrapped with three hoop FRP layers).
Figure 4.12: Steps of wrapping specimens in Group VHF (wrapped with one vertical layer then two hoop layers of FRP)
Figure 4.13: Steps of wrapping specimens in Group AHF (wrapped with two declined layers and then 1 hoop layer of FRP)

4.5 Preliminary testing

Preliminary testing included testing concrete cylinders, reinforcing bars and FRP sheets.
4.5.1 Concrete testing

Determination of compressive concrete strength was conducted according to the Australian Standards AS 1012.8 (2000) and AS 1012.9 (1999) for concrete cylinders. Three cylindrical specimens with 100 mm diameter and 200 mm height were cast from the same batch of concrete used to cast the column specimens. After the concrete was set, the cylinders were taken out of the molds and immersed into the curing tank to provide the specimens with enough moist to continue hydration and gain maximum strength. At the 28th day after casting, the specimens were tested to failure under compressive loading as shown in Figure 4.14.

![Figure 4.14: Testing of concrete cylinders](image)

To prevent premature cracking of concrete specimens under compressive loading, a high strength plaster was prepared with a plaster-to-water ratio of 3.5:1 (specified by producer to give an approximate compressive strength of 80 MPa) and used as capping to prevent premature cracking and to ensure an even transfer of load over the concrete sample loading surface during the test. Before testing average diameter, height and weight of each specimen was measured. The average 28 day concrete compressive strength was 38.2 MPa (Table 4.2).
### Table 4.2: Results of concrete testing

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Age (days)</th>
<th>Measured average diameter (mm)</th>
<th>Measured height (mm)</th>
<th>Load (kN)</th>
<th>Compressive strength (MPa)</th>
<th>Average compressive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>28</td>
<td>100</td>
<td>200</td>
<td>307</td>
<td>39.1</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>28</td>
<td>101</td>
<td>200</td>
<td>304</td>
<td>38.0</td>
<td>38.2</td>
</tr>
<tr>
<td>3</td>
<td>28</td>
<td>99.5</td>
<td>200</td>
<td>292</td>
<td>37.6</td>
<td></td>
</tr>
</tbody>
</table>

#### 4.5.2 Testing of reinforcing steel

The tensile testing method according to Australian Standard AS 1391 (AS 1391-2007 2007) was used to determine the reinforcing steel properties. Three samples of 6 mm diameter plain bar and three samples of 12 mm diameter deformed bar were taken from R6 transverse reinforcement bars and N12 longitudinal reinforcement bars. All samples were 550 mm length and were tested using the Instron 8033 tensile testing machine located in the Civil Engineering Laboratory at the University of Wollongong (Figure 4.15). Tensile load applied to the steel bars was monotonically increased up to the failure.

![Figure 4.15: Testing of steel bars](image)

The load applied (N) and corresponding deformation (mm) in the specimens were recorded via an electronic data acquisition system connected to the testing machine.
and were displayed in Figures 4.16 and 4.17. The average tensile strength and tensile strain of N12 were 587 MPa and 0.0029, respectively. Those for R6 were 538 MPa and 0.00267, respectively (Table 4.3).

**Figure 4.16**: Tensile test stress-strain curve of 12 mm diameter deformed bar

**Figure 4.17**: Tensile test stress-strain curve of 6 mm diameter deformed bar
Table 4.3: Reinforcing steel properties

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>R6 -S1</td>
<td>6</td>
<td>15227</td>
<td>1.068</td>
<td>539</td>
<td>0.00267</td>
<td>538</td>
<td>0.00267</td>
</tr>
<tr>
<td>R6 -S2</td>
<td>6</td>
<td>15173</td>
<td>1.058</td>
<td>537</td>
<td>0.00265</td>
<td>538</td>
<td>0.00269</td>
</tr>
<tr>
<td>R6 -S3</td>
<td>6</td>
<td>15180</td>
<td>1.077</td>
<td>537</td>
<td>0.00269</td>
<td></td>
<td></td>
</tr>
<tr>
<td>N12 - S1</td>
<td>12</td>
<td>64592</td>
<td>1.191</td>
<td>571</td>
<td>0.0030</td>
<td>567</td>
<td>0.00290</td>
</tr>
<tr>
<td>N12 - S2</td>
<td>12</td>
<td>64015</td>
<td>1.095</td>
<td>566</td>
<td>0.0027</td>
<td></td>
<td></td>
</tr>
<tr>
<td>N12 - S3</td>
<td>12</td>
<td>63809</td>
<td>1.103</td>
<td>564</td>
<td>0.0028</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

4.5.3 Testing of Carbon FRP sheets

The properties of FRP were determined from the FRP coupon tests according to (ASTM D7565/D7565M – 10 2010). Unidirectional CFRP fiber sheets (CARBON-UNI340GM-75MM) were used in this experimental program. The nominal width and thickness of the CFRP sheet were 75 mm and 0.45 mm, respectively. For unidirectional FRP sheets, ASTM D7565/D7565-10 (2010) specified the dimensions of the samples as shown in Figure 4.18. FRP coupons consisting of one, two and three parallel CFRP layers were prepared and tested (Figure 4.19). Multiple FRP layer wrapping was obtained by adhering the layers with epoxy resin. After the preparation of CFRP coupon samples, their actual dimensions were measured, such as width $w$, testing length $l_g$ (Table 4.4). Tensile load was applied by the Instron 8033 tensile testing machine in the High Bay Laboratory at the UOW (Figure 4.20).

![Figure 4.18: Dimensions of test FRP coupons](image-url)
**Table 4.4**: Test results of all FRP coupons

<table>
<thead>
<tr>
<th>Specimen</th>
<th>n (layers)</th>
<th>t (mm)</th>
<th>w (mm)</th>
<th>l_g (mm)</th>
<th>P_{max} (N)</th>
<th>δ_{max} (mm)</th>
<th>P_{(unit width)} (N/mm)</th>
<th>ε</th>
</tr>
</thead>
<tbody>
<tr>
<td>1H - S1</td>
<td>1</td>
<td>0.45</td>
<td>23.8</td>
<td>138</td>
<td>12605</td>
<td>1.93</td>
<td>530</td>
<td>0.0140</td>
</tr>
<tr>
<td>1H - S2</td>
<td>1</td>
<td>0.45</td>
<td>24.8</td>
<td>138</td>
<td>12903</td>
<td>1.87</td>
<td>520</td>
<td>0.0135</td>
</tr>
<tr>
<td>1H - S3</td>
<td>1</td>
<td>0.45</td>
<td>25.2</td>
<td>138</td>
<td>14187</td>
<td>2.03</td>
<td>563</td>
<td>0.0147</td>
</tr>
<tr>
<td>2H - S1</td>
<td>2</td>
<td>0.9</td>
<td>24.6</td>
<td>138</td>
<td>24352</td>
<td>1.96</td>
<td>990</td>
<td>0.0142</td>
</tr>
<tr>
<td>2H - S2</td>
<td>2</td>
<td>0.9</td>
<td>24.5</td>
<td>138</td>
<td>22698</td>
<td>1.93</td>
<td>926</td>
<td>0.0140</td>
</tr>
<tr>
<td>2H - S3</td>
<td>2</td>
<td>0.9</td>
<td>25.1</td>
<td>138</td>
<td>26287</td>
<td>2.10</td>
<td>1047</td>
<td>0.0152</td>
</tr>
<tr>
<td>3H - S1</td>
<td>3</td>
<td>1.35</td>
<td>25.1</td>
<td>138</td>
<td>35997</td>
<td>2.31</td>
<td>1434</td>
<td>0.0167</td>
</tr>
<tr>
<td>3H - S2</td>
<td>3</td>
<td>1.35</td>
<td>25.5</td>
<td>138</td>
<td>40554</td>
<td>2.50</td>
<td>1590</td>
<td>0.0181</td>
</tr>
<tr>
<td>3H - S3</td>
<td>3</td>
<td>1.35</td>
<td>25.2</td>
<td>138</td>
<td>34753</td>
<td>2.16</td>
<td>1379</td>
<td>0.0157</td>
</tr>
</tbody>
</table>
Figure 4.21: Force per unit width - strain diagrams for 1-layer FRP samples

Figure 4.22: Force per unit width - strain diagrams for 2-layer FRP samples

Figure 4.23: Force per unit width - strain diagrams for 3-layer FRP samples
The test results of all FRP coupons are given in Table 4.4. Force per unit width-strain diagrams for 1-, 2- and 3-layer samples are shown in Figures 4.21, 4.22 and 4.23, respectively. The properties of CFRP sheets are summarized in the Table 4.5 as below:

<table>
<thead>
<tr>
<th>Configuration</th>
<th>1 layer</th>
<th>2 layers</th>
<th>3 layers</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width, mm</td>
<td>24.6</td>
<td>24.7</td>
<td>25.3</td>
</tr>
<tr>
<td>Gage length, mm</td>
<td>138</td>
<td>138</td>
<td>138</td>
</tr>
<tr>
<td>Tensile force before failure, N</td>
<td>13232</td>
<td>24446</td>
<td>37101</td>
</tr>
<tr>
<td>Ultimate displacement, mm</td>
<td>1.94</td>
<td>2.00</td>
<td>2.32</td>
</tr>
<tr>
<td>Tensile force per unit width, N/mm</td>
<td>538</td>
<td>988</td>
<td>1468</td>
</tr>
<tr>
<td>Ultimate strain</td>
<td>0.0149</td>
<td>0.0145</td>
<td>0.0168</td>
</tr>
<tr>
<td>Tensile modulus of elasticity, MPa</td>
<td>85033</td>
<td>75926</td>
<td>64575</td>
</tr>
</tbody>
</table>

### 4.6 Test set up of specimens

Testing of the specimens was conducted at the High Bay laboratory, University of Wollongong using Denison 5000 kN compression testing machine. Four groups of specimens (RC, HF, VHF, and AHF) were divided into two series of specimens.

The first series consisting of twelve specimens were tested as columns under axial compression loading. The twelve specimens were divided into four groups with three specimens each. In each group of specimens, the specimens were tested at three different eccentricities of 0, 25, and 50 mm. These specimens were denoted by the corresponding group label accompanied with 0, 25, or 50 at the end. For example, Specimens RC-0, RC-25, RC-50 denotes that these specimens are in Group RC (reference columns) and are tested under eccentricities of 0, 25, and 50 mm, respectively (Table 4.6).

The second series consisting of four specimens were tested as beams under four-point loading regime. These specimens were denoted by the corresponding group label accompanied with B at the end i.e. RC-B, HF-B, VHF-B and AHF-B as shown in Table 4.6.
Table 4.6: Test set of column-tested specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Internal reinf.</th>
<th>Configuration (CFRP)</th>
<th>Eccentricity (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC-0</td>
<td></td>
<td></td>
<td>0</td>
</tr>
<tr>
<td>RC-25</td>
<td>4N12</td>
<td>None</td>
<td>25</td>
</tr>
<tr>
<td>RC-50</td>
<td>&amp; R6 @100</td>
<td></td>
<td>50</td>
</tr>
<tr>
<td>RC-B</td>
<td></td>
<td>pure bending</td>
<td></td>
</tr>
<tr>
<td>HF-0</td>
<td></td>
<td>Three</td>
<td>0</td>
</tr>
<tr>
<td>HF-25</td>
<td>4N12</td>
<td>hoop</td>
<td>25</td>
</tr>
<tr>
<td>HF-50</td>
<td>&amp; R6 @100</td>
<td>layers</td>
<td>50</td>
</tr>
<tr>
<td>HF-B</td>
<td></td>
<td>pure bending</td>
<td></td>
</tr>
<tr>
<td>VHF-0</td>
<td></td>
<td>One vertical</td>
<td>0</td>
</tr>
<tr>
<td>VHF-25</td>
<td>4N12</td>
<td>and two hoop</td>
<td>25</td>
</tr>
<tr>
<td>VHF-50</td>
<td>&amp; R6 @100</td>
<td>layers</td>
<td>50</td>
</tr>
<tr>
<td>VHF-B</td>
<td></td>
<td>pure bending</td>
<td></td>
</tr>
<tr>
<td>AHF-0</td>
<td></td>
<td>Two (±45°)</td>
<td>0</td>
</tr>
<tr>
<td>AHF-25</td>
<td>4N12</td>
<td>and one hoop</td>
<td>25</td>
</tr>
<tr>
<td>AHF-50</td>
<td>&amp; R6 @100</td>
<td>layers</td>
<td>50</td>
</tr>
<tr>
<td>AHF-B</td>
<td></td>
<td>pure bending</td>
<td></td>
</tr>
</tbody>
</table>

1 All specimens have the dimensions 200x200 mm in cross-section and 800 mm in height.

4.6.1 Testing of columns

Square, steel loading caps were used in testing all column-tested specimens. The loading cap consists of two parts: a 50 mm thick steel plate, called adaptor plate, which exerts the load from the loading ram to the column, and a 25 mm thick steel plate with a circular roller, which was used in eccentric loading tests. Details of the loading caps are shown in Figure 4.24 and Figure 4.26.

Figure 4.24: Details of loading caps
Figure 4.25: Plaster and placing a specimen into the loading cap

High strength plaster was placed in the caps (to create a level surface between the plate and the bottom or top ends of the specimens) and left to set for at least 45 mins before testing as shown in Figure 4.25. The bottom-loading cap was centered using a rig holding the column in place. A forklift was used to place the specimen in the testing machine. The top load cap was placed as the column was lifted into place. Photos of typical compression tests are shown in Figure 4.26.

In order to measure the lateral displacement of the columns, a laser triangulation sensor was placed horizontally at the front of the protective perspex shield with a small cutout hole for the laser. A data taker was used to link the testing machine with a computer to detect the loading ram that exerts the load, the longitudinal deflection, and the lateral deflection from the laser triangulation sensor. Testing commenced under displacement controlled condition. All tests were conducted at a rate of 0.3 mm/min.
Figure 4.26: Typical compression testing: (a) Concentric, (b) Eccentric
4.6.2 Testing of beams

Four specimens were tested as beams under four point loading regime. The span of the beam was 700 mm. An equipment consisting of two rigs, as shown in Figure 4.27, was used to exert the load generated by the loading ram of the Denison compression testing machine to the. The bottom rig was placed diagonally on the bottom plate of the machine and the beam specimen was then placed on the rig, followed by the top rig.

Figure 4.27: A typical beam testing
The axial deflection at the mid-span of the beam was measured at the bottom of the specimen using a laser triangulation sensor that was placed vertically underneath the bottom rig with a small cutout hole for the laser. A data taker was used to link the testing machine and the laser triangulation sensor to a computer to give the load from the loading ram and the mid-span deflection of the beam from the sensor, respectively. The beam was tested under displacement control, the end-point position was set at 50 mm, and the loading rate was set at 0.3 mm/min.

4.7 Summary

In order to investigate the effect of fiber orientations on hollow core RC members, an experimental program was designed and conducted at High Bay Laboratory, University of Wollongong. The experiment involved in testing twelve hollow core RC columns and four hollow core RC beams. Steps of design specimens, casting specimens, and wrapping them with FRP sheets were described in detail in this part of the thesis. Preliminary tests were conducted on concrete, steel bars and FRP sheets in order to determine their mechanical properties. Finally, testing equipment and steps of the test setup process was explained. The results of the experiments are presented and discussed in the next chapter.
5 RESULTS AND DISCUSSION

5.1 Introduction

This chapter presents results of testing all specimens. The chapter is divided into two parts.

In the first part, the results of testing the twelve column specimens are presented. These results are analyzed to investigate the behavior of unwrapped columns and FRP-confined columns in terms of strength, ductility and failure modes. After this, theoretical calculations are carried out to construct axial load and bending moment (P-M) interaction diagrams of each group of specimens. These theoretical interaction diagrams are compared to those obtained from the experimental results for all groups of specimens.

The second part presents results of testing the four beam specimens. Flexural and shear behaviors of the beams are analyzed and evaluated. Theoretical calculations using existing models for predicting the shear strength capacity of the strengthened beams are conducted and compared with experimental results.

5.2 Behavior of column-tested specimens

Table 5.1 shows results of testing all columns. Ultimate displacement for unwrapped specimens was taken at the point when the corresponding axial load was equal to 85% of maximum load, and for FRP-confined, the ultimate displacement was taken at the point when the FRP ruptured. Similar to solid reinforced concrete column, external wrapping of CFRP can enhance the performance of hollow core RC columns, in terms of strength and ductility. The gain in strength is significant for columns wrapped exclusively with hoop FRP layers (HF columns) under concentric loading or when the eccentricity of the load is small (25 mm). When the eccentricity is large (50 mm), the strength gain is more considerable for columns with the presence of vertical or ±45° CFRP wraps (VHF and AHF columns). Further, these HF columns showed a greater ductility than unwrapped columns, and the columns in groups VHF and AHF.
Table 5.1: Summary of testing results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max. load</th>
<th>Corresponding displacement at max. load</th>
<th>Ultimate displacement</th>
<th>Corr. axial load at ultimate displacement</th>
<th>Normalized max. load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kN</td>
<td>Axial mm</td>
<td>Lateral mm</td>
<td>Axial mm</td>
<td>Lateral mm</td>
</tr>
<tr>
<td>RC-0</td>
<td>1341</td>
<td>3.32</td>
<td>N/A</td>
<td>3.58</td>
<td>N/A</td>
</tr>
<tr>
<td>HF-0</td>
<td>1485</td>
<td>21.99</td>
<td>N/A</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>VHF-0</td>
<td>1525</td>
<td>4.03</td>
<td>N/A</td>
<td>8.07</td>
<td>N/A</td>
</tr>
<tr>
<td>AHF-0</td>
<td>1417</td>
<td>3.81</td>
<td>N/A</td>
<td>7.02</td>
<td>N/A</td>
</tr>
<tr>
<td>RC-25</td>
<td>998</td>
<td>2.99</td>
<td>1.71</td>
<td>3.15</td>
<td>1.99</td>
</tr>
<tr>
<td>HF-25</td>
<td>1245</td>
<td>3.75</td>
<td>1.48</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>VHF-25</td>
<td>1189</td>
<td>3.80</td>
<td>1.57</td>
<td>8.17</td>
<td>6.14</td>
</tr>
<tr>
<td>AHF-25</td>
<td>1083</td>
<td>3.73</td>
<td>2.29</td>
<td>7.60</td>
<td>7.01</td>
</tr>
<tr>
<td>RC-50</td>
<td>755</td>
<td>3.06</td>
<td>2.20</td>
<td>3.38</td>
<td>2.70</td>
</tr>
<tr>
<td>HF-50</td>
<td>825</td>
<td>3.67</td>
<td>2.56</td>
<td>26.96</td>
<td>27.94</td>
</tr>
<tr>
<td>VHF-50</td>
<td>889</td>
<td>4.13</td>
<td>3.23</td>
<td>10.32</td>
<td>8.34</td>
</tr>
<tr>
<td>AHF-50</td>
<td>862</td>
<td>3.78</td>
<td>2.50</td>
<td>8.79</td>
<td>6.65</td>
</tr>
</tbody>
</table>

1 The lateral displacement is not applicable in cases of concentric loading
2 Data was lost due to an accident during testing

5.2.1 Group RC

Figure 5.1 shows the failure modes of columns in Group RC, reference columns.

![Figure 5.1: Failure modes of RC columns tested under 0, 25 and 50 mm](image-url)
These columns did not suffer any large strain after reaching their maximum load, but failed in a brittle manner characterized by peeling off the concrete and outward buckling of the steel bars within two adjacent ties in the compression side for all columns. The failure positions were about 200 mm from the ends as shown in Figure 5.1. Horizontal cracks were also found in the tension side at the failure position of columns under eccentric testing. The columns showed the highest load carrying capacity of 1341 kN when tested under concentric loading. When eccentricities were introduced, the maximum load carrying capacity decreased significantly. Columns RC-25 and RC-50 showed maximum loads of 998 kN and 755 kN that were 25.5% and 43.7% reduction, respectively. This reduction in the axial load is due to moments resulting from the eccentricity and the second-order lateral deflections of the column that can be recognized through theoretical and experimental P-M interaction diagrams (Figures 5.9-5.13).

### Table 5.2: Level of decrease in the maximum load of eccentrically tested columns as compared to the corresponding concentrically tested columns

<table>
<thead>
<tr>
<th>Column</th>
<th>Maximum load (kN)</th>
<th>Normalized maximum load (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC-0</td>
<td>1341</td>
<td>1.0</td>
</tr>
<tr>
<td>RC-25</td>
<td>998</td>
<td>0.74</td>
</tr>
<tr>
<td>RC-50</td>
<td>755</td>
<td>0.56</td>
</tr>
<tr>
<td>HF-0</td>
<td>1485</td>
<td>1.0</td>
</tr>
<tr>
<td>HF-25</td>
<td>1245</td>
<td>0.84</td>
</tr>
<tr>
<td>HF-50</td>
<td>825</td>
<td>0.56</td>
</tr>
<tr>
<td>VHF-0</td>
<td>1525</td>
<td>1.0</td>
</tr>
<tr>
<td>VHF-25</td>
<td>1189</td>
<td>0.78</td>
</tr>
<tr>
<td>VHF-50</td>
<td>889</td>
<td>0.58</td>
</tr>
<tr>
<td>AHF-0</td>
<td>1417</td>
<td>1.0</td>
</tr>
<tr>
<td>AHF-25</td>
<td>1083</td>
<td>0.76</td>
</tr>
<tr>
<td>AHF-50</td>
<td>862</td>
<td>0.61</td>
</tr>
</tbody>
</table>

5.2.2 Group HF

Wrapping columns with three layers in the hoop orientation was the most efficient method for increasing the strength and ductility of columns for both concentric and eccentric loading. Specimen HF-0 showed 11% increase in maximum load compared to the reference specimen RC-0 in concentric testing. Specimens HF-25 and HF-50 achieved 25% and 9% increases, respectively compared with the corresponding
Specimen HF-0 was expected to gain the highest applied load of the four columns tested concentrically, however a premature failure was observed. After examining the tested HF-0 specimen carefully, it was found that the concrete had spalled at a corner at the surface of the top end where the longitudinal steel bar at that corner was exposed. It is supposed that longitudinal steel bar had moved upward during casting this specimen leading to the concrete cover at the top of this longitudinal steel bar was just 4 mm instead of the required 20 mm. This unexpected upward movement of the steel bar might be the reason for this exposure as it caused high stress concentration at this corner when the load was applied. This premature failure only occurred to HF-0 (Figure 5.2).

**Figure 5.2:** Premature damage at a corner of Specimen HF-0

In terms of ductility, Figures 5.3 to 5.5 show clearly that hoop layers can substantially extend axial displacement as well as lateral displacement of the columns under concentric and eccentric loading. Table 5.3 shows the ductility calculation for all specimens. In the present study, ductility was calculated based on axial displacement behavior which was suggested by GangaRao et al. (2007). That is the ductility is equal to the ratio of the axial displacement at 85% maximum load (post-yielding point) $\Delta_{85\%P_{max}}$ and the axial displacement at yield load $\Delta_y$, i.e. ductility = $\Delta_{85\%P_{max}} / \Delta_y$. Hoop orientation wrapping of Specimen HF-0 increased ductility 4.7 times than that of the reference specimen RC-0, and around 2.4 times those of Specimens VHF-0 and AHF-0. Under eccentric loading, these comparisons
are even larger. Specimen HF-50 showed ductility 7.4 times larger than Specimen RC-50 and about 3.4 times larger than Specimens VHF-50 and AHF-50. This significant high ultimate displacement of HF-50 is due to the contribution of axial load is still larger than that of bending effect. According to theoretical calculations, at the eccentricity of 50 mm, the neutral axis depth of the specimen was 166.3 mm (out of 200 mm of cross-section depth) resulting in compression dominated the section. This theoretical prediction was confirmed by the experimental result where the failure point of Specimen HF-50 occurred in compression failure zone (Refer to Figure 5.11 and Table 5.4). The ductility of Specimen HF-25 was not calculated due to an accident that occurred with the machine as testing commenced. The applied load suddenly increased with a very high-speed and in about 16 seconds, 1250 kN was applied before the test was stopped. The load data recorded from the computer showed that the applied load reached the maximum value and then started decreasing. It was decided to start the test again, but some cracks were already found on the FRP layers. The maximum load the column achieved was only about 80% of the previous value recorded during the accident. The axial and lateral displacements however, were still very large as shown in Figure 5.4. Therefore, CFRP layers in the hoop direction were efficient in delaying premature failure of columns due to the spalling of concrete and buckling of steel bars at yield load.
**Figure 5.3:** Load-axial displacement curves for concentrically loaded columns

**Figure 5.4:** Load-displacement curves for 25 mm eccentrically loaded columns. Note: HF-25 (1) and HF-25 (2) refer to 1st and 2nd loading periods of Specimen HF-25 due to an accident during test.
Figure 5.5: Load-displacement curves for 50 mm eccentrically loaded columns

Table 5.3: Calculation of ductility

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Axial displacement (mm)</th>
<th>Ductility at yield load</th>
<th>Ductility at 85% $P_{max}$</th>
<th>Normalized ductility at 85% $P_{max}$ / $\Delta y$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\Delta y$</td>
<td>$\Delta 85% P_{max}$</td>
<td>$\Delta 85% P_{max}$ / $\Delta y$</td>
<td></td>
</tr>
<tr>
<td>RC-0</td>
<td>2.80</td>
<td>3.58</td>
<td>1.3</td>
<td>1.0</td>
</tr>
<tr>
<td>HF-0</td>
<td>3.83</td>
<td>22.98</td>
<td>6.0</td>
<td>4.7</td>
</tr>
<tr>
<td>VHF-0</td>
<td>2.92</td>
<td>8.05</td>
<td>2.8</td>
<td>2.2</td>
</tr>
<tr>
<td>AHF-0</td>
<td>2.75</td>
<td>6.76</td>
<td>2.5</td>
<td>1.9</td>
</tr>
<tr>
<td>RC-25</td>
<td>2.62</td>
<td>3.15</td>
<td>1.2</td>
<td>1.0</td>
</tr>
<tr>
<td>HF-25</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>VHF-25</td>
<td>2.92</td>
<td>6.07</td>
<td>2.1</td>
<td>1.7</td>
</tr>
<tr>
<td>AHF-25</td>
<td>3.05</td>
<td>5.87</td>
<td>1.9</td>
<td>1.6</td>
</tr>
<tr>
<td>RC-50</td>
<td>2.59</td>
<td>3.38</td>
<td>1.3</td>
<td>1.0</td>
</tr>
<tr>
<td>HF-50</td>
<td>2.78</td>
<td>26.89</td>
<td>9.7</td>
<td>7.4</td>
</tr>
<tr>
<td>VHF-50</td>
<td>2.78</td>
<td>8.52</td>
<td>3.1</td>
<td>2.3</td>
</tr>
<tr>
<td>AHF-50</td>
<td>2.94</td>
<td>7.89</td>
<td>2.7</td>
<td>2.1</td>
</tr>
</tbody>
</table>

*Data was lost due to an accident during testing*

It is worthy to note that when the eccentricity was large, the gain in strength of HF columns decreased, whereas the gain in ductility increased. This result is due to the contribution of bending is greater than axial load that caused premature failure in the specimen (refer to Figure 5.10). FRP in this wrapping method only works in the hoop direction. As such there was no effect in preventing the loss of strength caused by such bending moment. Meanwhile, bending creates hoop strain in the compression fiber of the section. This strain was restrained by the FRP in the hoop direction, which allowed the specimen to achieve large deformation capacity.

Specimens of Group HF failed in a sudden, catastrophic manner. When the applied load increased to large enough values, small cracking sounds were heard. After the maximum load, the load carrying capacity of the columns slightly reduced while the deflection significantly increased allowing HF columns to achieve large ductility. When the deflection was large enough, the cracking sounds increased and the FRP ruptured with a loud explosive noise as it reached its ultimate value. The failure position of FRP layers of Specimens HF-0 and HF-50 was determined to be about 160 mm - 200 mm from the top end and that for Specimen HF-25 was near the mid-
height of the column. All of these FRP layers ruptured at a corner of the column due to stress concentration at corners of non-circular cross-section columns. Failure modes of all specimens are shown in Figure 5.6.

![Image of FRP column failure modes](image)

**Figure 5.6**: Failure modes of HF columns tested under 0, 25 and 50 mm

5.2.3 Groups VHF and AHF

The behaviors of VHF and AHF columns were quite similar in both concentric and eccentric loading, especially with regard to ductility. Both of these wrapping schemes improved the ductility of the columns. The combination of two ±45° oriented CFRP layers and one hoop CFRP layer of AHF columns were expected to show the largest ductility compared to the other schemes, but their gain in ductility was only around 1.6 to 2.1 times larger than the corresponding reference specimens (Table 5.3). In the case of concentric loading, similar results were also report in the Sadeghian et al.’s study (2010) which was conducted on solid plain concrete cylinders that the combination of transverse and angle oriented layers did not allow the specimens to undergo large axial deflections. Therefore, it might be concluded that this type of wrapping combination is not effective in enhancing ductility and energy absorption for concentrically loaded columns. More tests are needed to verify
this conclusion. Meanwhile, Sadeghian et al. (2010) confirmed a significant increase in ductility for cylinders wrapped with only $\pm 45^\circ$ orientations. In the case of eccentric loading, the gain in the specimens’ deformation was not significant. Possible reason is that the contribution of axial loading is more than that of bending in this experiment.

With respect to strength, as expected, the gain in strength of these specimens increased when the eccentricity increased due to the contribution of the vertical and $\pm 45^\circ$ wraps. When the columns were tested concentrically, there was no contribution of vertical and angle layers resulting in only a slight increase in the strength, which in fact came from the hoop layers (Table 5.1). At an eccentricity of 50 mm, the contribution of vertical and angle layers became clearer. The increase in strength of Specimens VHF-50 and AHF-50 were even greater than that of Specimen HF-50 (i.e. 9% for HF-50, meanwhile 18% and 14% for VHF-50 and AHF-50, respectively). Therefore, the presence of vertical and angle CFRP orientation is clearly efficient in resisting bending moments due to eccentricities that caused the premature failure of RC columns. This achievement is similar to Hadi (2007) and Hadi and Widiarsa (2012) in testing solid reinforced concrete columns when vertical FRP layers were added.

Specimens in Groups VHF and AHF also failed in a sudden and explosive manner. When the applied load was large enough, small cracking sounds were heard. A bulging deformation on the outer hoop FRP layer in the compression side was also observed. After the maximum load, the load carrying capacity of VHF and AHF specimens reduced significantly. The rupture of the outer hoop layers caused a loud explosive noise and the load dropped.

All VHF columns failed at a position approximately 130 mm from the top end (Figure 5.7). With the exception of Specimen VHF-0, the rupture of FRP in VHF specimens (VHF-25 and VHF-50) occurred in the middle, on the side of the column, not at the corner. After examining inside the hole of these specimens, it was found that the concrete at the inner corners broke in a way that tends to make the cross-section of the hole at failure position circular. As a result, the concrete in the middle
of the compression side expanded outwards causing the FRP failure near the middle side of the column.

**Figure 5.7:** Failure modes of VHF columns tested under 0, 25 and 50 mm

**Figure 5.8:** Failure modes of AHF columns tested under 0, 25 and 50 mm
For AHF columns, the rupture of FRP started from the corner and further developed into an approximate 45° downward angle in the compression side of the column. The failure positions varied between 130 mm to 200 mm from the bottom end of the columns (Figure 5.8).

All the specimens failed near the ends, not the desired midspan. Possible reason is the stress concentration at the boundaries.

5.2.4 Comparison between theoretical calculations and experimental results

In order to predict the axial loading capacity and combined axial compression and flexure, P-M interaction diagrams were drawn based on theoretical calculations for both unconfined RC columns and FRP-confined columns.

For unconfined RC columns, P-M diagrams can be developed by satisfying strain compatibility and force equilibrium using the model for the stress-strain behavior of conventional RC columns. The detailed procedure is based on (Warner et al. 2007).

A similar procedure was applied in this study to construct the P-M diagram for the FRP-confined specimens. However, the stress-strain model for unconfined concrete was replaced by a stress-strain model for FRP-confined concrete.

The axial load capacity under concentric loading is computed by using Eq. (5.1):

\[ N_{uo} = f'_{cc} A_{net} + f_y A_s \]  

where \( f'_{cc} \) is the compressive strength of FRP-confined concrete. The calculation of \( f'_{cc} \) was presented in chapter 3, section 3.2.2; \( f_y, A_s \) are the yield strength and area of longitudinal steel reinforcement, respectively. For columns with the presence of vertical or ±45°-oriented layers, it was assumed that there is no contribution of these layers in the confinement of the concrete core under concentric loading. These contributions are only taken into account in the cases of eccentric loading, where combined axial compression and flexure took place.

The axial load capacity \( N_u \) and bending moment \( M_u \) under eccentric loading are computed by using Eqs. (5.2) and (5.3):
\[ N_u = (C_c + C_s) - \left( T + T_{frp} \right) \]  
(5.2)

\[ M_u = C_c (d_{pc} - Y_c) + S_c (d_{pc} - d_{sc}) + T (d - d_{pc}) + T_{frp} (d_{frp} - d_{pc}) \]  
(5.3)

where \( C_c, C_s \) are the compressive forces in concrete and longitudinal steel reinforcement, respectively; \( T, T_{frp} \) are tensile forces in the tensile steel reinforcement and FRP, respectively; \( d_{pc}, d_{sc}, d, d_{frp} \) are distances from the extreme compression concrete fiber to the plastic centroid, centroid of compressive steel reinforcement, centroid of tensile steel reinforcement, and centroid of the FRP, respectively; and \( Y_c \) is the centroid of concrete in the compression region.

Rectangular stress block was used to compute the compressive force of concrete in the compression region, because at the point where the strain in tensile steel reinforcement is equal to zero, the neutral axis depth \( d_n \) at failure is at the level of the tensile reinforcement. Further, at the balanced failure condition, the neutral axis depth \( d_n \) is also observed to be very close to the compressive flange of the cross-section that do not contain a large hollow part.

The strain in FRP \( \varepsilon_{frp} \) was computed using strain compatibility, which was controlled by both confined concrete axial strain \( \varepsilon_{cc} \) at maximum stress \( f'_{cc} \), and effectively hoop rupture strain of FRP \( \varepsilon_{fe} \).

\[ \varepsilon_{frp} = \varepsilon_{cc} \left( \frac{d_{frp} - d_n}{d_n} \right) \leq \varepsilon_{fe} \]  
(5.4)

in which, \( d_{frp} \) is the distance from the extreme compression fiber to the neutral axis, taken equal to the cross-section depth \( h \).

The theoretical calculation results for all specimens are given in Table 5.4. In Table 5.4, the first bending moment capacity \( M_{I \, theo.} \) was computed by multiplying the theoretical maximum axial load capacity \( P_{theo.} \) and the corresponding eccentricity \( (e) \). Because the tested specimens were considered to be nonsway pin-ended columns subjected to equal moments at the two ends, theoretical secondary bending moment \( (M_{II \, theo.}) \) was computed by multiplying the first moment \( M_{I \, theo.} \) induced by the
eccentricity and a moment magnifier $\delta_{ns}$, as given in Eq. 5.5 as follows:

$$M_{II \text{theo}} = \delta_{ns} M_{I \text{theo}}. \tag{5.5}$$

For pin-ended columns moment magnifier $\delta_{ns}$ is computed by the following equation:

$$\delta_{ns} = \frac{1}{1 - P / P_E} \tag{5.6}$$

In which, $P$ is the total axial load that is product of the dead load and the applied load; $PE$ is the Euler buckling load given by Eq. 5.7:

$$P_E = \frac{\pi^2 EI}{l^2} \tag{5.7}$$

$$EI = \frac{0.2E_c I_g + E_s I_{sc} + E_{frp} I_{frp}}{1 + \beta_{dns}} \tag{5.8}$$

Where, $l$ is length of the column; $EI$ is the flexural rigidity of column cross-section computed by Eq. 5.8; $E_c$, $E_s$, $E_{frp}$ are moduli of elasticity of the concrete, steel reinforcement and CFRP reinforcement, and their corresponding moment of inertia $I_g$, $I_{sc}$, and $I_{frp}$, respectively.

The term $(1 + \beta_{dns})$ reflects the effect of creep on the column deflections. Due to the specimens were tested in short time in the experimental program, the term $(1 + \beta_{dns})$ is taken to be equal to 1.

For experimental bending moment values, $M_{I \text{expt.}}$ was computed by multiplying the maximum applied load ($P_{expt.}$) and the corresponding eccentricity ($e$). Secondary bending moment ($M_{II \text{expt.}}$) was computed by using Eq. (5.5) as follows:

$$M_{II} = P_{max} (e + \delta) \tag{5.9}$$

where $\delta$ is the corresponding lateral deflection at the maximum applied load $P_{expt.}$, which are obtained from the experimental lateral deflection records.
Table 5.4: Summary of theoretical calculations

<table>
<thead>
<tr>
<th>Group</th>
<th>Ecc. max. load ($P_{theo}$)</th>
<th>Theo. bending moment $M_{I_{theo}} = \frac{P_{theo}}{P_{theo}}$</th>
<th>Theo. secondary moment $M_{II_{theo}} = \frac{\delta_{ns}M_{I_{theo}}}{\delta_{ns}}$</th>
<th>Exp. max. load ($P_{expt}$)</th>
<th>$M_{I_{expt}} = \frac{P_{expt}}{P_{expt}}$</th>
<th>$M_{II_{expt}} = \frac{P_{expt}(e+\delta)}{P_{expt}}$</th>
<th>Difference $\frac{P_{expt} - P_{theo}}{P_{theo}} \times 100$</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC</td>
<td>0</td>
<td>1274</td>
<td>0</td>
<td>0</td>
<td>1341</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>962</td>
<td>24.1</td>
<td>26.14</td>
<td>998</td>
<td>25.0</td>
<td>26.7</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>741</td>
<td>37.1</td>
<td>39.78</td>
<td>755</td>
<td>37.7</td>
<td>39.4</td>
</tr>
<tr>
<td>HF</td>
<td>0</td>
<td>1535</td>
<td>0</td>
<td>0</td>
<td>1485</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>1132</td>
<td>28.3</td>
<td>31.05</td>
<td>1245</td>
<td>31.1</td>
<td>33.0</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>890</td>
<td>44.5</td>
<td>48.17</td>
<td>825</td>
<td>41.3</td>
<td>43.4</td>
</tr>
<tr>
<td>VHF</td>
<td>0</td>
<td>1452</td>
<td>0</td>
<td>0</td>
<td>1525</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>1076</td>
<td>26.9</td>
<td>29.20</td>
<td>1189</td>
<td>29.7</td>
<td>31.6</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>845</td>
<td>42.3</td>
<td>45.34</td>
<td>889</td>
<td>44.4</td>
<td>47.3</td>
</tr>
<tr>
<td>AHF</td>
<td>0</td>
<td>1366</td>
<td>0.00</td>
<td>0</td>
<td>1417</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>1018</td>
<td>25.5</td>
<td>27.47</td>
<td>1083</td>
<td>27.1</td>
<td>29.6</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>795</td>
<td>39.8</td>
<td>42.45</td>
<td>862</td>
<td>43.1</td>
<td>45.2</td>
</tr>
</tbody>
</table>

Figure 5.9 shows the theoretical P-M interaction diagrams of all groups of specimens. It can be seen that with the assumption of no contribution of vertical or angle oriented FRP layers under concentric loading, the gain in strength of columns depend on the thickness of hoop FRP layers. The increase in the number of hoop FRP layers can result in an increase in the compression strength capacity of the columns subjected to concentric loads. The test results were of the same tendency (refer to Table 5.1). The theoretical pure bending moments of RC specimens and HF specimens were equal, while those for VHF and AHF specimens were significantly greater than RC and HF specimens due to the contribution of the vertical and the 45°-oriented layers, which are close to the tensile steel reinforcement.

Figures 5.10 to 5.13 show comparisons of the P-M interaction diagrams between the theoretical calculation and the experimental results for the four groups of specimens.
Figure 5.9: Theoretical calculation of P-M interaction diagrams

Figure 5.10: Theoretical and experimental P-M interaction diagrams for columns in Group RC
Figure 5.11: Theoretical and experimental P-M interaction diagrams for columns in Group HF

Figure 5.12: Theoretical and experimental P-M interaction diagrams for columns in Group VHF
From Figures 5.10 to 5.13, it can be concluded that the theoretical calculations showed good agreement with the experimental results. With the exception of Specimens HF-0 and HF-50, the theoretical results underestimated the experimental results for all of the specimens. For unwrapped specimens there was about 2% to 5% difference between the theoretical and the experimental results. VHF specimens and AHF specimens showed differences of 5.0% to 10.5% and 3.8% to 8.4%, respectively. Specimen HF-0, which experienced premature failure during the concentric testing process showed -3.2 % difference from the theoretical prediction and that for Specimen HF-50 was -7.3% (the minus sign here denotes the experimental results were lower than the calculated results). Specimen HF-25 showed 10% difference compared with the theoretical calculation.

The theoretical strain results of reinforcement were also compared with the data obtained from strain gages showing good predictions. Details of arrangement of strain gages are given in Figure 4.3.
Table 5.5 shows the strain of longitudinal steel bars from strain gages and compares these with the theoretical calculations at maximum load for all of the specimens under concentric loading, 25 and 50 mm eccentricities. For concentric loading, the theoretical strain of steel bars was assumed to have yielded and was equal to 2900µε. The experimental strain was taken by the average value of four strain gages glued on the four longitudinal steel bars. The strain of Specimen RC-0 showed 8.8% difference compared to the theoretical calculation, meanwhile Specimens VHF-0 and AHF-0 were 14% and 15.4%, respectively. The longitudinal steel bars strain of Specimen HF-0 was not applicable here because of the variation due to premature failure.

In the case of eccentric loading, the theoretical strains of longitudinal steel bars were carried out for the steel bars on the tension side and compression side based on the principles of strain compatibility and were controlled by concrete strain at maximum load $\varepsilon_{cc}$ and the effective strain in FRP $\varepsilon_{ef}$. The strains in the steel bars on the compression side of unwrapped Specimens RC-25 and RC-50 showed about 27% difference compared to the theoretical calculation, while those differences for Specimens HF and VHF were between 12% and 20%. Specimens AHF-25 and AHF-50 showed differences of 32% and 27.2%, respectively. The strain in the steel bars on tension side showed very good agreement with the theoretical calculation in the case of 50 mm eccentric loading. At this eccentricity, the theoretical strain of the tensile steel layer is equal to zero. In the case of 25 mm eccentric loading, the strain

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**Figure 4.3:** Location of strain gages at mid-height of the column (repeated page 35)
in the tensile steel bars showed large differences compared to the theoretical calculations. These strain values in fact were very small, which may have been caused by inaccuracy.

**Table 5.5: Theoretical and experimental strains of longitudinal steel bars at maximum load**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Strain of bars in compression side (µε)</th>
<th>Strain of bars on tension side (µε)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(Average value of SG3 and SG4)</td>
<td>(Average value of SG1 and SG2)</td>
</tr>
<tr>
<td></td>
<td>Theo.</td>
<td>Exp.</td>
</tr>
<tr>
<td>RC-0</td>
<td>2900</td>
<td>3156</td>
</tr>
<tr>
<td>RC-25</td>
<td>2420</td>
<td>3073</td>
</tr>
<tr>
<td>RC-50</td>
<td>2390</td>
<td>3044</td>
</tr>
<tr>
<td>HF-0</td>
<td>2900</td>
<td>N/A</td>
</tr>
<tr>
<td>HF-25</td>
<td>3880</td>
<td>3149</td>
</tr>
<tr>
<td>HF-50</td>
<td>3730</td>
<td>4302</td>
</tr>
<tr>
<td>VHF-0</td>
<td>2900</td>
<td>3305</td>
</tr>
<tr>
<td>VHF-25</td>
<td>3124</td>
<td>3501</td>
</tr>
<tr>
<td>VHF-50</td>
<td>3080</td>
<td>3698</td>
</tr>
<tr>
<td>AHF-0</td>
<td>2900</td>
<td>3346</td>
</tr>
<tr>
<td>AHF-25</td>
<td>2621</td>
<td>3477</td>
</tr>
<tr>
<td>AHF-50</td>
<td>2550</td>
<td>3243</td>
</tr>
</tbody>
</table>

1The experimental strains of bars SG1, SG2 at 50 mm eccentric loading were very close to zero. Note: Refer to Fig. 12 for locations of Strain Gages SG1 – SG4

The strain in a tie at mid-height of the column in both tension and compression sides were also measured for all columns, and are given in Table 5.6. It is to be noted that in cases of concentric loading or when the eccentricity was small (25 mm), the strain in the ties of all the specimens was significant at maximum load, but the ties had not yet yielded. Therefore, it is necessary to take the contribution of ties into account in calculating the strength of confined concrete in the cases where the stress-strain curves are of the descending type and the ties were still working. When the eccentricity was large (50 mm), the strain of ties decreased considerably as shown in Table 5.5. The first possible reason is due to the actual location of failure of the tested specimens far away from the location of placing Strain Gages. Stress redistribution took place after the most critical failure position established, resulted in the most critical strain at this critical failure position. The strain of ties at this failure position would be the highest strain value as compared to strain of ties at other positions. Another cause might be the effect of bending. Buckling due to the
eccentricity caused premature failure to occur in the specimens that prevented a tie to achieve its higher strain values. The fact is that up to the yield point of the steel, no differences exist in the confinement mechanism between the steel and the FRP as they both behave in a linear elastic way. The less strain in ties may hint the less strain in the outer FRP layers. This in turn explains the inefficiency of FRP under large eccentric loading where premature failure due to bending moment dominates the failure modes of the specimens. In addition, at the tension side when the strain of longitudinal steel bars were close to zero, the strain of tie at that side was also close to zero.

Table 5.6: Strain of the tie at mid-height for all of specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>At yield load ($\mu$e)</th>
<th>At max. load ($\mu$e)</th>
<th>At 85% of max. load ($\mu$e)</th>
<th>At load corr. to ultimate strain ($\mu$e)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SG5</td>
<td>SG6</td>
<td>SG5</td>
<td>SG6</td>
</tr>
<tr>
<td>RC-0</td>
<td>752</td>
<td>563</td>
<td>1261</td>
<td>770</td>
</tr>
<tr>
<td>HF-0</td>
<td>1129</td>
<td>450</td>
<td>1861</td>
<td>1599</td>
</tr>
<tr>
<td>VHF-0</td>
<td>913</td>
<td>593</td>
<td>1413</td>
<td>935</td>
</tr>
<tr>
<td>AHF-0</td>
<td>960</td>
<td>639</td>
<td>1915</td>
<td>1234</td>
</tr>
<tr>
<td>RC-25</td>
<td>130</td>
<td>786</td>
<td>135</td>
<td>1153</td>
</tr>
<tr>
<td>HF-25</td>
<td>N/A</td>
<td>N/A</td>
<td>158</td>
<td>1426</td>
</tr>
<tr>
<td>VHF-25</td>
<td>79</td>
<td>-3</td>
<td>91.6</td>
<td>-</td>
</tr>
<tr>
<td>AHF-25</td>
<td>82</td>
<td>696</td>
<td>101</td>
<td>1356</td>
</tr>
<tr>
<td>RC-50</td>
<td>13</td>
<td>592</td>
<td>13</td>
<td>813</td>
</tr>
<tr>
<td>HF-50</td>
<td>10</td>
<td>520</td>
<td>19</td>
<td>856</td>
</tr>
<tr>
<td>VHF-50</td>
<td>8</td>
<td>531</td>
<td>19</td>
<td>820</td>
</tr>
<tr>
<td>AHF-50</td>
<td>0</td>
<td>546</td>
<td>2</td>
<td>749</td>
</tr>
</tbody>
</table>

1At that point the strain gage already reached limit strain value (1% for strain gages glued on tie) and failed.
2The strain gage values were not applicable because Specimen HF-25 got accident during testing.
3The strain gage broke.
Note: Refer to Fig. 12 for locations of Strain Gage SG5 and SG6
5.3 Behavior of beam specimens

Four specimens that were tested under four-point loading test in order to investigate the effect of the FRP fiber orientation on the shear performance of hollow RC beams. Details of the specimens were described in Section 4.2, and are also shown in Figure 5.14. The first specimen, which was not wrapped with CFRP, served as the control specimen. The other three specimens were completely wrapped with continuous unidirectional CFRP sheets around the cross-section of the specimens, as described in Section 4.4. For convenience during the analysis, the wrapping schemes of these specimens are shown more detail in Table 5.7 and Figure 5.15. The test results of all beams are given in Table 5.8.

![Figure 5.14: Details and dimensions of beam-tested specimens](image)

<table>
<thead>
<tr>
<th>Beam</th>
<th>Wrapping scheme</th>
<th>Loading condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC-B</td>
<td>None</td>
<td>Four-point loading</td>
</tr>
<tr>
<td>HF-B</td>
<td>0°/0°/0°</td>
<td>Four-point loading</td>
</tr>
<tr>
<td>VHF-B</td>
<td>90°/0°/0°</td>
<td>Four-point loading</td>
</tr>
<tr>
<td>AHF-B</td>
<td>45°/45°/0°</td>
<td>Four-point loading</td>
</tr>
</tbody>
</table>

1 All specimens are 800 mm length, 200 mm x 200 mm square section with a hollow of 80 mm x 80 mm inside
### Table 5.8: Beams test results

<table>
<thead>
<tr>
<th>Beam</th>
<th>Applied load (kN) yield</th>
<th>Applied load (kN) max.</th>
<th>Corr. mid-span deflection (mm) at 85%P&lt;sub&gt;max&lt;/sub&gt; yield load</th>
<th>Corr. mid-span deflection (mm) at 85%P&lt;sub&gt;max&lt;/sub&gt; max. load</th>
<th>Corr. mid-span deflection (mm) at 85%P&lt;sub&gt;max&lt;/sub&gt; 85%P&lt;sub&gt;max&lt;/sub&gt;</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC-B</td>
<td>N/A&lt;sup&gt;2&lt;/sup&gt;</td>
<td>156</td>
<td>132</td>
<td>N/A</td>
<td>3.1</td>
<td>13.6</td>
</tr>
<tr>
<td>HF-B</td>
<td>198</td>
<td>308</td>
<td>262</td>
<td>3.8</td>
<td>27.6</td>
<td>33.7</td>
</tr>
<tr>
<td>VHF-B</td>
<td>240</td>
<td>327</td>
<td>278</td>
<td>4.2</td>
<td>12.2</td>
<td>12.4</td>
</tr>
<tr>
<td>AHF-B</td>
<td>232</td>
<td>498</td>
<td>423</td>
<td>3.9</td>
<td>20.1</td>
<td>21.3</td>
</tr>
</tbody>
</table>

1. The applied load at the point where the tensile steel bars started yielding
2. At that point the tensile steel bars had not yet yielded.

### Figure 5.15: Details of fiber orientation combinations

5.3.1 Failure behaviors and crack modes

In this study, the tested beams are regarded as short shear span beams (or deep beams) since the shear span-to-effective depth ratio \( a/d \) was 1.42 (Figure 5.16). For short shear span beams with shear span-to-effective depth \( a/d \) from 1.0 to 2.5, the behavior of the beams will be governed by arch action (Wight and MacGregor 2012).
Therefore, their shear behavior can be explained in accordance with the arch action. That is, after inclined cracks were initiated, the concrete along the potential inclined shear cracks serve as inclined compression struts, and the steel reinforcement serve as tension ties. This arch action is able to carry additional load after the beam develops inclined cracks.

5.3.1.1 Control beam

The control beam displayed shear-tension failure under a four-point loading test, as shown in Figure 5.17. As the applied load was increased to the crack load, the beam developed inclined cracks joining the support and the loading point. These inclined cracks caused the shear capacity of the beam to drop below the flexural capacity leading to shear failure dominated flexural failure. The development of the inclined cracks was observed as follows:

At the first stage, very small inclined cracks were observed at a load of 137 kN (No pictures had been taken at this stage). There was one crack at the left shear span and another crack at the right shear span of the beam. These inclined cracks initially appeared at the center of the shear spans and quickly propagated simultaneously both downward to the supports at the level of the tension steel bars and upward to the loading points at the level of the compression steel bars. There was no evident of flexural cracks. It was observed that at this stage, the applied load was still
increasing. This increase in the applied load may be due to both the activation of stirrups in preventing inclined cracks and the arch action.

At the next stage, with the increase of the load, the cracks opened wider. When the width of the cracks became clear as shown in the Figure 5.17a, the applied load reached its maximum value of 156 kN then started decreasing. The cracks started to propagate deeper into the soffit of the beam. Finally, the applied load gradually decreased, while the cracks opened more severely with the increase in the deflection of the beam at mid-span. More inclined cracks were also developed at the right shear span of the beam as the shown in Figure 5.17b when the deflection was larger.

In summary, the propagation of the inclined shear cracks of the control beam can be divided into three stages. At the first stage, very small inclined cracks, between the level of compression steel bars and tension steel bars, joining the support and the load were developed. However, the applied load was still increasing due to the activation of the stirrups and the arch action in deep beams. During the next stage,
the applied load reached its maximum and then started decreasing. The opening of the cracks at the maximum applied load was clear. In the third stage, as the load was continually applied, the width of the critical shear inclined cracks opened wider. Splitting cracks and inclined cracks along the critical inclined cracks developed.

5.3.1.2 CFRP-wrapped beams

The failure modes of CFRP-wrapped beams are shown in Figure 5.18, 5.19 and 5.20. The change of the CFRP fiber direction in the combinations resulted in different failure modes of specimens. Specimen HF-B which was wrapped with three hoop FRP layers showed flexural failure, while Specimen AHF-B which was wrapped with two diagonal (±45°) layers and one hoop CFRP layer failed in shear. Specimen VHF-B which was wrapped with one longitudinal and two hoop CFRP layers experienced premature failure at the supports.

The failure mode and cracking propagation of Specimen HF-B was similar to the conventional RC beams that failed in flexure, as shown in Figure 5.18a. When the applied load was large enough, small cracking sounds were heard. These sounds may be due to the CFRP straining in tension when the inclined cracks opened. The deflection of the beam increased with the increase in the applied load. The gaps between the adjacent CFRP sheets opened clearer and wider exposing vertical cracks on the concrete surface. The width of these vertical cracks was wide at the midspan and the cracks towards both ends had cracks with less width. The beam reached its maximum applied load of 308 kN, and soon after failed by the rupture of both the two tensile steel bars near the middle of the beam at the center of a CFRP sheet. The rupture of tensile steel bars occurred suddenly with a loud explosive noise. The load dropped totally, and the beam could not carry any further load. After the CFRP cover was removed, shear cracks were also observed (Figure 5.18b). Therefore, it can be concluded that at the beginning of the loading stage, the inclined shear cracks were developed. Due to the contribution of three hoop CFRP layers, the shear capacity of the beam was substantially strengthened, which was then higher than flexural capacity of the beam. With the increase of the applied load, vertical flexural cracks developed that dominated the failure mode of the beam while the propagation of the inclined shear cracks was controlled.
Specimen VHF-B experienced premature failure at the supports (Figure 5.19). When the load was at around 290 kN, cracks were observed at the two ends of the beam. The applied load then reached its maximum value very soon after at the load of 327 kN. However, this maximum applied load was still two times higher than that of the control specimen. This significant increase in the applied load of Specimen VHF-B indicates that CFRP sheets in this orientation combination significantly enhanced the shear capacity of the beam. The presence of one longitudinal CFRP layer at the soffit of the beam along the beam’s axis also contributed to flexural strength of the beam. Consequently, the beam was strengthened in both the shear strength and flexural strength, which became greater than the carrying capacity of the beam at the supports. High stress concentration at the supports caused premature failure of the beam. Therefore, with a hollow beam, it is important to check the carrying capacity of the beam at the supports after flexural and shear strengthening of the beam is complete.

![Flexural cracks](image1.png)

![After the CFRP cover was taken out](image2.png)

**Figure 5.18**: Failure mode of Specimen HF-B
Figure 5.19: Failure mode of Specimen VHF-B

a) Only shear cracks were observed at the left end because of the cover of CFRP

b) after the CFRP cover was taken out

c) shear cracks were observed at the right end of the beam after removing the FRP

Figure 5.20: Failure mode of Specimen AHF-B
With the experience from the premature failure at the supports suffered by Specimen VHF-B, the two ends of Specimen AHF-B were strengthened by adding high-strength plaster of about 100 mm length making them solid, as shown in Figure 5.21. The test result showed that Specimen AHF-B failed in shear in a sudden and brittle manner. This failure was caused by the rupture of CFRP sheets along the inclined crack, which joins the loading point and the support at the left shear span of the beam (Figure 5.20a). The inclined crack in the CFRP did not occur at the right shear span of the beam. However, when the CFRP cover of the specimen was taken out, inclined shear cracks in the concrete at the right end joining the loading point and the support was still observed (Figure 5.20b, c). Therefore, the reason for this phenomenon does not arise from the impact of the high-strength plaster part, otherwise may be due to errors during the test set-up process or from the specimen that caused an unequal distribution of the applied load at the two loading points. In this case, the levels at the loading points between the two ends might not be equal leading to the left end received a much larger amount of the applied load as compared to the right end. Hence the CFRP at the left end was reached its ultimate rupture capacity before the CFRP in the right end.

Figure 5.21: Strengthening two ends of Specimen AHF-B with high-strength plaster

The propagation of the inclined crack in Specimen AHF-B was observed as follows: When the applied load was at about 423 kN, continuous small cracking sounds were heard from the CFRP. When the applied load reached a load of 498 kN, the upper CFRP sheets at the left shear span ruptured causing a loud explosive noise. The load dropped to 400 kN, then continued dropping to about 280 kN with another loud
explosive noise due to the rupture of the lower CFRP sheets. The CFRP sheets ruptured along the inclined crack from the loading point to the left support as shown in Figure 5.20a.

5.3.2 Load-deflection behaviors

Figure 5.22 shows the axial load-midspan deflection for all tested beams. It can be seen clearly that the four tested specimens had almost the same modulus of elasticity at the initial loading stage. They only started being different after the formation of shear cracks or yielding of tensile steel reinforcement.

Generally, all strengthening schemes significantly enhanced the strength and midspan deflection capacity at the maximum load of the hollow RC beam. Of the four tested specimens, only the control specimen and Specimen AHF-B failed in shear. These two specimens are then taken into a consideration for shear strengthening with CFRP sheets.

![Figure 5.22: Axial load-midspan deflection diagrams of all beams](image)

The control specimen showed a maximum applied load of 156 kN (Figure 5.22). Meanwhile, that for Specimen AHF-B was 498 kN, which was 219% increase in the
shear strength as compared with the control specimen, respectively. These results show that the CFRP sheets in this wrapping scheme can substantially increase the shear strength and the deflection capacity of the hollow RC beam.

After reaching the maximum applied load, the midspan deflection of the control specimen still showed good ductility (Figure 5.22). This ductility is due to the presence of stirrups and the tensile steel reinforcement. In contrast, Specimen AHF-B failed in a sudden, explosive manner due to the rupture of the CFRP after the maximum load was reached. This failure occurred without warnings and was catastrophic and brittle. The specimen lost almost 50% of its load carrying capacity after the CFRP ruptured.

Three hoop CFRP sheets changed the failure mode of Specimen HF-B to flexural failure. This type of failure is more ductile than shear failure, and as a result, allowed the specimen to undergo larger deflection while the applied load was still increasing. Specimen HF-B had the corresponding midspan deflection of 27.6 mm at the maximum applied load, 9 times larger than that of the control beam. Moreover, after the tensile steel bars yielding, both the applied load and the deflection of the beam at midspan was increased, which led to an ascending axial load-deflection curve for Specimen HF-B (Figure 5.22).

<table>
<thead>
<tr>
<th>Beam</th>
<th>Mid-span deflection at (mm) yield load</th>
<th>85%Pmax</th>
<th>Ductility</th>
<th>Normalized ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(Δy)</td>
<td>(Δ85%Pmax)</td>
<td>Δ85%Pmax / Δy</td>
<td></td>
</tr>
<tr>
<td>RC-B</td>
<td>3.1*</td>
<td>13.6</td>
<td>4.4</td>
<td>1.00</td>
</tr>
<tr>
<td>HF-B</td>
<td>3.8</td>
<td>33.7</td>
<td>9.0</td>
<td>2.04</td>
</tr>
<tr>
<td>VHF-B</td>
<td>4.2</td>
<td>12.4</td>
<td>3.0</td>
<td>0.68</td>
</tr>
<tr>
<td>AHF-B</td>
<td>3.9</td>
<td>21.3</td>
<td>5.4</td>
<td>1.23</td>
</tr>
</tbody>
</table>

* The value was taken at position of the maximum applied load for calculation since all ties of Specimen RC-B had not yielded

Although Specimen VHF-B was prematurely damaged at the supports, the applied load reached a maximum load of 327 kN, an increase of 109% as compared to the control beam. The midspan deflection of the beam was 12.2 mm at the maximum load.
Ductility calculation was conducted for all beam specimens and is given in Table 5.9. Figure 5.23 displays the ductility comparison of FRP-wrapped specimens to the control specimen. HF-B obtained the largest ductility, which is about 2 times larger than that of the control beam. AHF-B showed a 23% increase in ductility as compared to control beam. Meanwhile, due to the bearing premature failure, VHF-B did not show any significant deformation capacity, but failed in a brittle manner.

![Ductility comparison between beam specimens](image)

**Figure 5.23:** Ductility comparison between beam specimens

5.3.3 Strain at longitudinal steel reinforcement

In the control beam, which failed in shear, the tensile steel bars did not yield at the maximum load, as shown in Table 5.9. In contrast, the tensile steel bars in all CFRP-wrapped beams had yielded. This larger strain in the tensile steel bars of the CFRP-strengthened beams explains their greater deflection capacity as compared with the control beam.

After yielding, the tensile steel bars in all the strengthened beams were strained dramatically. However, these strains at the maximum load of the beams were reduced. This reduction of these strains at the maximum load may be due to the modes of failure and the positions of failure, which were formed in the beams.
Table 5.10: Strain in the tensile steel bars

<table>
<thead>
<tr>
<th>Beam</th>
<th>Strain in the tensile steel bars (µε)</th>
<th>at yield load</th>
<th>max. value of strain between yield load and max. load</th>
<th>at max. load</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC-B</td>
<td>did not yield</td>
<td></td>
<td></td>
<td>2122</td>
</tr>
<tr>
<td>3V-B</td>
<td>2900</td>
<td>10285</td>
<td></td>
<td>3274</td>
</tr>
<tr>
<td>1H2V-B</td>
<td>2900</td>
<td>12357</td>
<td></td>
<td>7415</td>
</tr>
<tr>
<td>2A1V-B</td>
<td>2900</td>
<td>10256</td>
<td></td>
<td>6892</td>
</tr>
</tbody>
</table>

For Specimen HF-B the position of the rupture of the tensile steel bars was out of the middle of the beam. Therefore, when the most critical position formed, the steel bars were mostly strained at that position. Meanwhile, the Strain Gages were glued at the middle of the beam. Consequently, the strain data of the Strain Gages at that position reduced. For the Specimen VHF-B, because the premature failure occurred at the supports, the strain in the tensile steel bars reduced. For Specimen AHF-B, at the beginning both flexural and shear strength were developed with the increase of the applied load. As a result, the strain in the tensile steel bars was increased. Because the shear strength capacity was less than the flexural strength capacity, the shear failure dominated. The development of the inclined cracks reduced the bending capacity of the beam causing the strain in the tensile steel bars to decrease.

5.3.4 Comparison between theoretical calculations and experimental results

At the present time, no specific models for predicting the shear capacity of short shear span beams strengthened with FRP composites have been adopted by the design standards for the FRP, such as ACI code, Euro code. Therefore, three models developed by Triantafillou and Antonopoulos (2000), Khalifa et al. (1998), and Chen and Teng (2003) were adapted to calculate the contribution of the CFRP to the shear strength of the strengthened specimens used in the present study. The reason for choosing these models is because they have been widely accepted in the literature.

The calculation of shear capacity was carried out on Specimen AHF-B, which failed in shear. The theoretical results are given in Table 5.10. The contribution of the CFRP $V_f$ to the shear strength of the specimen is given in the second column of the table, which was calculated according to Khalifa et al. (1998), Triantafillou and
Antonopoulos (2000), and Chen and Teng (2003). The details of the calculation process were presented in Section 3.3. The combined shear resistance of the concrete and stirrups \( V_c + V_s \) was determined from the experimental results of the control beam. It was taken as the shear resistance of the control beam at the deflection corresponding to the maximum applied load of the strengthened beam (AHF-B), i.e. \( V_c + V_s = \text{Applied load}/2 = 96.5/2 = 48.3 \text{ kN} \). Hence, the shear strength of Specimen AHF-B is the sum of \( V_c + V_s \) and \( V_f \), which is given in the fourth column of the table.

Table 5.11: Theoretical calculations for Specimen AHF-B

<table>
<thead>
<tr>
<th>Reference</th>
<th>( V_f ) (kN)</th>
<th>( V_c+V_s ) (kN)</th>
<th>( V_{\text{theo}} ) (kN)</th>
<th>( V_{\text{exp}} ) (kN)</th>
<th>Diff. (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Khalifa et al. (1998)</td>
<td>173.2</td>
<td>48.3</td>
<td>221.5</td>
<td>249</td>
<td>12.4</td>
</tr>
<tr>
<td>Triantafillou and Antonopoulos (2000)</td>
<td>128.0</td>
<td>48.3</td>
<td>176.3</td>
<td>249</td>
<td>41.3</td>
</tr>
<tr>
<td>Chen and Teng (2003)</td>
<td>173.0</td>
<td>48.3</td>
<td>221.3</td>
<td>249</td>
<td>12.5</td>
</tr>
</tbody>
</table>

Note: \( V_f \) is the shear contribution of CFRP, \( V_c+V_s \) is the combined shear contribution of concrete and stirrups.

Table 5.10 shows that all of the three models underestimated the shear strength capacity of Specimen AHF-B. Of the three models, the models of Khalifa et al. (1998) and Chen and Teng (2003) were in good agreement with the experimental results. Both models showed about 12% difference between the theoretical calculation and experimental results. Meanwhile, the model of Triantafillou and Antonopoulos (2000) was more conservative than the former two models. Triantafillou and Antonopoulos’ model (2000) showed 41.3% difference when compared with the experimental result.

### 5.4 Summary

In this chapter, the test results of all specimens were presented and evaluated. From the test results, it is found that the FRP fiber orientation has a significant effect on the performance of hollow rectangular RC columns under axial loading. Three combinations of CFRP fiber orientations were used in this experiment. Among these fiber orientation combinations, the combination of three hoop CFRP layers was shown to be the most efficient method for enhancing the strength and ductility of the
hollow square RC columns. The enhancement in ductility of the columns was significant for both concentric and eccentricity loading. This enhancement in ductility was even greater with the increase in eccentricity. Meanwhile, the enhancement in strength was significantly reduced with the increase in eccentricity. The two other combinations of fiber orientation, which consisted of vertical and ±45° oriented CFRP layers, were shown to be viable to resist these combined actions. In fact, the contribution of vertical and diagonal CFRP layers was more evident with the increase in eccentricity. However, the ductility enhancement in these two combinations was not noticeable. All CFRP-wrapped columns showed axial load-displacement curves of the descending type. Hence, the stress-strain model developed by Mander et al. (1988) was used in this study to predict the strength of the confined columns. The results from theoretical calculations were in good agreement with the experimental results.

This chapter also presents the results of testing four hollow beams under a four-point loading test. The results revealed that CFRP sheets could substantially increase the shear performance of hollow RC beams. The change in fiber directions resulted in different failure modes of the beams. Three hoop CFRP sheets changed the failure mode of the control beam, which failed in shear, to the flexural failure which is more ductile than shear failure. The combination of both diagonal and hoop layers considerably increased both shear strength and flexural strength of Beam AHF-B, which failed in shear by the rupture of CFRP sheets along the inclined crack joining the loading point and the support. The propagation of the inclined cracks in a hollow beam with a short shear span-to-effective depth $a/d$ ratio was also fully analyzed through observing the failure mode of the control beam RC-B.

In the next chapter, the main conclusions of the present study are summarized, following by recommendations for future research.
6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

This thesis involves testing sixteen specimens in order to experimentally study the effect of FRP fiber orientation on the performance of hollow core square RC columns/beams externally strengthened with CFRP sheets. Three combinations of different fiber orientations (0°, 45°, and 90°) were used to wrap the specimens. The first series of twelve specimens were tested as columns under both concentric and eccentric loading. The second series of four specimens were tested as beams under four-point loading. The failure modes, the gain in strength and ductility of all specimens were observed. From the analysis of the test results, the following conclusions can be drawn:

1. All of the three wrapping combinations used in this study increased the strength and ductility of the hollow core square RC columns. The enhancement in ductility was more evident than the enhancement in strength, in particular for columns wrapped with only hoop oriented layers.

2. The enhancement in the compressive strength of CFRP-wrapped hollow core columns is marginal. All CFRP-wrapped specimens used in this study showed only a 9 - 25% increase in strength as compared with the unwrapped specimens.

3. The fiber in the hoop direction can significantly increase the ductility of the hollow core square RC columns under both concentric and eccentric loading. Compared to VHF and AHF columns, HF columns can sustain 2.9 – 4.6 times larger deformation before failure. On the other hand, the enhancement in strength of the columns wrapped with these hoop layers was only evident when the columns were tested concentrically or when the eccentricity of the load is small. When the columns were loaded under large eccentricity, the enhancement in strength was significantly reduced.

4. In this case, when the eccentricity of the load is large, the contribution of vertical and ±45° oriented layers was clear. This contribution was more
noticeable as the eccentricity increased. In fact, Specimens AHF-50 and VHF-50, which were respectively wrapped with one and two hoop CFRP layers gained maximum axial load even greater than that of Specimen HF-50, which was wrapped with three CFRP layers in the hoop direction. There is no contribution of these two wrap orientations on the strength of columns under concentric loads.

5. In this study, the combination of ±45° oriented CFRP layers and one hoop layer did not show any significant increase in deflections of the columns under both concentric and eccentric testing. In fact, the behavior of the specimen wrapped with this wrapping scheme was nearly similar to the specimens wrapped with the combination of vertical and hoop layers. Both these two combinations showed 1.6 - 2.4% enhancement in ductility as compared with the control columns. Possible reason is that the contribution of axial loading is more than that of bending. Also, it appears that the presence of one outer hoop layer might prevent the deflection of the ±45° layers causing the premature failure of the CFRP layers in this combination.

6. Although Mander et al.’s stress-strain model (1988) is for solid rectangular RC columns, the theoretical calculations using this model for predicting strength of tested specimens were in good agreement with the experimental results. The selection of Mander et al.’s model (1988) in this paper was based on the shape of the stress-strain curves of the tested specimens. In the authors’ opinion, when a column shows a stress-strain curve of ascending type, which is similar to that of Mander et al.’s model (1988), the behavior of that column is similar to specimens studied by Mander et al. (1988).

7. CFRP sheets can substantially increase the shear strength of the hollow RC beams. CFRP-wrapped specimens showed a 97% to 219% increase in shear strength as compared with control beam.

8. The change in the fiber direction strongly affected the gain in the shear capacity and failure modes of the beam specimens. The combination of vertical CFRP layers to the longitudinal axis of the beam changed the failure mode of
the beam to flexural failure, which was significant ductility than shear failure. The combination of two diagonal and one vertical CFRP sheets to the longitudinal axis of the beam considerably increase the shear strength and deflection capacity of the beam.

6.2 Recommendations for future research

Possible future research areas are suggested as followed:

1. The present study limits the investigation of fiber orientation effects on testing square hollow cross-section columns. Section shape may affect the effectiveness of the fiber orientations on the behavior of the specimen. Therefore, different section shapes such as rectangular, circular or ellipse could be done in the future study. From that the effect of this section shape factor may be assessed.

2. It is well-known that the axial load and moment capacities are significantly affected by length and loading conditions of the column. If the slenderness ratio of a column is large, the failure may not occur by crushing, but by buckling. Buckling failure prevents the column from attaining its full strength because of instability. In the present study, only short columns were designed to be tested. More works are suggested on testing taller columns in order to investigate the effectiveness of the fiber orientation on the performance of these columns, in which the slenderness effect is taken into consideration. Different loading conditions such as different larger eccentricities, can also be further required.

3. The four hollow beams used in this study were all strengthened by using complete wrapping method. More experimental works can be conducted on different wrapping methods such as side bonding, U-shape wrapping in the form of continuous or discrete strips.

4. There have been no specific models for predicting CFRP-confined hollow columns and short shear span CFRP-strengthened beams at the present time. More theoretical analyses and experimental works should be done to enable the
prediction of such performance.
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APPENDIX

The experimental results of all specimens of the experimental program are given in the compact disk accompanied with the thesis.