Behaviour of fibre-reinforced polymer (FRP) tube reinforced concrete (FTRC) specimens under different loading conditions

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Behaviour of fibre-reinforced polymer (FRP) tube reinforced concrete (FTRC) specimens under different loading conditions

By

Ali Qasim Luaibi AL-BAALI

A thesis is submitted in partial fulfilment of the requirements for the reward of the degree of

MASTERS OF ENGINEERING (Research)

January 2016
Thesis Declaration

I, Ali Qasim Luaibi AL-Baali, declare that this thesis, submitted in partial fulfilment of the requirements for the ward of Master of Engineering (Research), in the school of Civil, Mining and 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.

Ali Qasim Luaibi AL-BAALI

January 2016
To my Father, Mother, Brothers and my Sister
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Publications

Conference Paper

Abstract

This study provides an experimental investigation of the behaviour of fibre-reinforced polymer (FRP) tube reinforced concrete (FTRC) specimens under different loading conditions. Four groups of 16 specimens with diameter of 200 mm and height of 800 mm were cast and tested. Specimens in the first group (Group REF) which is reference group were reinforced with longitudinal steel bars and steel helixes. Specimens in the second group (Group ST) were reinforced with intact glass FRP tubes. Specimens in the third group (Group ST-G) were also reinforced with intact glass FRP tubes. In addition, polymer grid was embedded into the concrete cover to reduce the cover spalling. Specimens in the fourth group (Group PT) were reinforced with perforated glass FRP tubes to integrate concrete cover with concrete core. From each group, one specimen was tested under concentric loading, one specimen under 25 mm eccentric loading, one specimen under 50 mm eccentric loading, and one specimen under four-point loading. Results from the experimental study show that FRP tubes significantly increase the load carrying capacity of FTRC specimens. Group ST-G specimens performed better than the other groups of specimens. Axial load-bending moment (P-M) Interaction diagrams constructed based on the experimental results also show the enhanced performance of FTRC specimens.

In addition, the P-M behaviour of steel reinforced concrete specimens SRCs and concrete filled FRP tube specimens CFFT's is analysed theoretically. An equivalent rectangular stress block method is used for SRCs and layer by layer method is used for CFFT's. A comparison between the theoretical P-M behaviour and experimental P-M behaviour is carried out for Group REF, Group ST, and Group ST-G separately. In general, the experimental and theoretical P-M interaction diagrams exhibit the same patterns except for Group REF.
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Chapter 1 Introduction

1.1 Overview

Most of the existing concrete columns are reinforced or confined with steel materials; while there have been a few applications of using fibre reinforced polymer (FRP) composite materials as a reinforcement or confinement for concrete columns. For concrete structures that have been constructed in aggressive environments such as marine environment, they can be deteriorated in a short period due to low corrosion resistance of steel reinforcement. As result, rehabilitation and repair of the concrete members can cost a lot of money.

In recent years, a considerable number of studies have been conducted to investigate the effect of using fibre reinforced polymer (FRP) composite materials on the behaviour of concrete structures. FRP materials have received much attention to use them for strengthening existing or new concrete structures due to many advantages such as high strength, high corrosion resistance and high stiffness. FRP composite material can be found in different configurations such as bars, sheets and tubes. FRP bars can be used as reinforcement for concrete columns instead of traditional steel reinforcement but their compressive strength is low in comparison with steel bars. In addition, for strengthening existing concrete columns, FRP sheets can be wrapped around the columns. Also, FRP tubes and sheets can be used as external confinement for newly constructed concrete columns. FRP sheets and FRP tubes can significantly enhance the strength and ductility of concrete columns.

The majority of the previous studies about using FRP tubes for strengthening concrete columns were focused on using them as external confinement. According to Bisby et al. (2005) and Ji et al. (2008), FRP sheets or tube confined concrete columns have a low resistance to high temperature. Also, the failure of FRP confined concrete column may
happen without indication such as cover cracks or spalling. Due to the above reasons, the application of FRP tube and FRP sheet as confinement of concrete has been limited.

1.2 Significance

In this study, FRP tube reinforced concrete column (FTRC) is proposed as a new composite column Hadi et al. (2015). In FTRCs, FRP tube is used as longitudinal reinforcement and transverse confinement for concrete columns. Furthermore, the concrete cover for FTRCs can reduce the negative effect of the high temperature on FRP tube, and the spalling of concrete cover can be used as an indication before the sudden failure of concrete column.

In order to investigate the behaviour of FTRCs under different loading conditions, an experimental program was carried out at High Bay civil engineering laboratory at University of Wollongong, Australia. A total sixteen concrete specimens were cast and tested under concentric, eccentric, and flexural loadings. The specimens were divided into four groups; each group consisted of four specimens. The first group (Group REF) was a reference group which was reinforced with steel bars and helix. The second group (Group ST) consisted of concrete specimens reinforced with solid GFRP tube. The third group (Group PT) consisted of concrete specimens reinforced with perforated GFRP tube. The last group (Group ST-G) was reinforced with GFRP tube. In addition, polymer grid was placed between the GFRP tube and concrete surface. The axial load-deformation, axial load-lateral deflection, and axial load-midspan deflection behaviours were examined. The failure modes, load carrying capacity, and ductility were also inspected. Finally, axial load-bending moment ($P-M$) interaction diagrams were constructed to examine the axial load and bending moment capacity of FTRC members.
1.3 Objectives

The main objectives of this study are as follows:

1. Conduct an Experimental program to investigate the behaviour of FTRC specimens under concentric, eccentric and flexural loadings and compare the behaviour with steel reinforced concrete (SRC) specimens.

2. Investigate the load carrying capacity, ductility, and axial load-bending moment behaviour of FTRCs and compare them with the results of SRC specimens.

3. Examine the effect of eccentricity on the load carrying capacity of FTRCs and compare it with SRCs.

4. Construct a theoretical axial load- bending moment \( (P-M) \) diagram to examine \( P-M \) behaviour of FTRC specimens and SRC specimens and compare the experimental \( P-M \) with the theoretical \( P-M \).

1.4 Arrangement of the thesis

This study consists of six chapters which are arranged as follows:

Chapter one is an introduction chapter which presents an overview, significance and the objectives of this study. In addition, the arrangement of the thesis is described in this chapter.

Chapter two presents a summary of previous studies related to FRP tube reinforced concrete specimens such as FRP tube confined concrete columns and steel tube reinforced concrete columns. In addition, the stress-strain model for both FRP and steel confined concrete are presented.

Chapter three describes the experimental program of this study. Firstly, the materials testing are described. Secondly, detailed preparation of the formwork and specimens are presented. Finally, casting of the specimens and the curing of the specimens are described.
Chapter four presents the analysis of the experimental results regarding the load carrying capacity, ductility, $P-M$ behaviour, and eccentricity effect of the FTRC specimens.

Chapter five provides a theoretical analysis of the $P-M$ behaviour by using layer by layer method for FTRC specimens and stress block method for SRC specimens. The comparisons between theoretical and experimental results from chapter four are presented.

Chapter six is the conclusion part of this study. Moreover, recommendations for future studies are presented.
Chapter 2 Literature review

2.1 Introduction

Recently, many studies have been conducted to investigate the effect of using fibre reinforced polymer (FRP) composite materials on the behaviour of concrete structures. In this chapter, a summary of existing studies related to the FRP tube-reinforced concrete columns is provided. Studies about FRP bars-reinforced concrete columns, steel tube-reinforced concrete columns, FRP reinforced concrete beams, FRP tube-confined concrete columns, and steel tube confined concrete columns are summarized and presented. In addition, a review of studies about confinement models for FRP and steel reinforced concrete columns are reviewed. To conclude, this chapter presents a summary of experimental and theoretical investigations related to the FRP tube reinforced concrete columns.

2.2 Behaviour of FRP bars reinforced concrete columns

Deiveegan and Kumaran (2011b) experimentally investigated the behaviour of GFRP bars reinforced concrete columns under concentric and eccentric loading. A total of forty-eight specimens were divided into two series, each series had twenty-four specimens were cast and tested. Different parameters were investigated in this study, such as, shape of the column, reinforcement ratio, and type of GFRP reinforcement, slenderness of the columns and concrete grades. Test results indicated that the specimens with the increased GFRP reinforcement ratio had the same behaviour as steel reinforced specimens. In comparison with steel reinforced specimens, the yield load of the GFRP bars was insignificant, and the deflection increased with the increased load. Brittle tension failure was exhibited in the columns reinforced with low GFRP reinforcement ratio.

Lotfy (2011) experimentally examined the axial behaviour of small scale FRP bars reinforced concrete columns. The experimental program consisted of eight square concrete columns
with dimensions of 250 mm by 250 mm and 1250 mm length, seven were GFRP reinforced concrete columns and closed steel stirrups with different diameters and one was steel reinforced specimens and closed stirrups. The main parameters investigated in this study were the main reinforcement ratio, transverse reinforcement ratio and characteristic strength of concrete. Test results showed that the behaviour of the GFRP reinforced concrete columns such as the ductility affected significantly by the increased main and transverse reinforcement ratios at the column ends and the increased characteristic strength of concrete. In addition, the increased transverse reinforcement ratio of the GFRP reinforced concrete columns increased the toughness and the ductility of the specimen. Finally, the ductility of the steel reinforced specimen was higher than that of GFRP reinforced specimens.

De Luca et al. (2011) investigated the behaviour of full-scale GFRP reinforced concrete columns subjected to axial load. The aim was to check the impact of GFRP bars behaviour on the compressive strength and failure mode. Furthermore, to check the influence of GFRP ties on concrete confinement and the prevention of instability of the longitudinal bars. The experimental program consisted of square full scale GFRP reinforced concrete columns with side dimension of 610 mm and 3000 mm length. Test results showed that the GFRP reinforced concrete columns had the same behaviour as steel reinforced concrete specimens and failure mode was significantly affected by tie spacing. For the evaluation of the nominal capacity of the columns, GFRP bars contribution can be neglected, because their contribution was less than 5% of the peak load.

Deiveegan and Kumaran (2011a) studied the behaviour of the full scale size GFRP reinforced concrete columns under eccentric loading. The experimental program consisted of forty-eight concrete columns with different dimensions and 2200 mm length divided into two series A
and B. Different parameters were studied in this paper, shape of column, reinforcement ratio, and type of the GFRP reinforcement, slenderness of the column and grades of the concrete. The experimental results showed that two types of failure modes were observed in GFRP reinforced concrete columns, crushing of the concrete and rupture of the GFRP bars in tension and compression. In addition, Series A and B failed by the rupture of the GFRP reinforcement in compression when their reinforcement ratios were 2.26% for rectangular columns and 3.8% for circular columns.

The strength and axial performance of CFRP bars and helix reinforced circular concrete columns were studied by Afifi et al. (2014b). Reinforcement ratio, longitudinal CFRP bars ratio and volumetric ratio, size, and spacing of helix were the test parameters used by the authors. Eleven specimens were used in this study, nine were reinforced with CFRP bars and CFRP helix, one specimen made of plain concrete, and the last one was steel reinforced concrete column. The steel reinforced and plain concrete specimens were used as a reference in this study. The dimensions of the columns were 300 mm diameter and 1500 mm height. This study indicated that, up to peak load, the steel reinforced and the CFRP reinforced concrete column had a similar behaviour. The columns with the higher longitudinal reinforcement ratio or with good confinement had a higher load carrying capacity. In addition, the failure mechanism of CFRP reinforced concrete columns with small volumetric ratio or large helix spacing was due to formation of strongly defined shear failure plane. Furthermore, the failure mechanism of well confined or small to moderate spacing of helix for CFRP reinforced concrete columns was due to rupture of helix and crushing of the concrete core. The CFRP reinforced concrete columns with 80 mm helix spacing had the same axial load-strain behaviour as steel reinforced concrete columns. For the ductility index, CFRP reinforced concrete columns sustained nearly 96% of the ductility that sustained by
steel reinforced concrete columns. The higher volumetric ratio specimens or the specimens with closer CFRP helix spacing had a higher ductility. On the other hand, the specimens with larger helix spacing had brittle behaviour and their strength decay after the peak was faster. The ductility and confinement efficiency were affected more by the amount of the CFRP helix than strength capacity.

Afifi et al. (2014a) experimentally investigated the axial capacity of GFRP bars and helix reinforced concrete columns with circular cross section. Providing better understanding and technical information about the compression behaviour of GFRP reinforced concrete columns are the main goals for this study. Twelve axially loaded full scale columns with dimensions of 300 mm diameter and 1500 mm height were used in this study, nine were GFRP bar and helix reinforced concrete columns, two were steel reinforced concrete columns and one was plain concrete specimen. The specimens were reinforced with Longitudinal GFRP bars and GFRP helix which is newly developed. Different parameters were investigated in this study such as, reinforcement type, longitudinal GFRP reinforcement ratio, volumetric ratio, diameters, and spacing of helix reinforcement. Test results indicated that using small diameter of GFRP helix with closer spacing have significant effect on the ductility and confinement efficiency rather than using GFRP helix with a large diameter and greater spacing. The axial load strain behaviour of GFRP reinforced concrete with helix spacing 40 mm – 80 mm was similar to the steel reinforced columns counterpart. The steel reinforced concrete columns were more ductile and showed higher rate of strength decay after peak compared to the GFRP reinforced concrete columns. Low reinforcement ratio specimens showed ductile and brittle behaviour compared to the specimens with high reinforcement ratio, which means that the reinforcement ratio effects significantly on the ductility and confinement efficiency. In addition, increased ratio of the longitudinal reinforcement from
1.1% to 3.2% reduced the peak load strain by 20%, strain of the longitudinal reinforcement by 25% and the peak strain of the transverse reinforcement by 86%. On the other hand, the strength was increased by 6% as the ratio of longitudinal reinforcement increased from 1.1% to 3.2%. In general, the confinement efficiency and ductility were highly affected by the GFRP helix reinforcement ratio than the strength capacity.

Mohamed et al. (2014) studied the performance of axially loaded concrete columns longitudinally reinforced with sand coated FRP bars and confined with sand coated FRP hoops and helix. Fourteen full scale concrete columns with dimensions of 300 mm diameter and 1500 mm height were used in this study, twelve were GFRP or CFRP bars and helix reinforced concrete columns, one plain concrete column and the last one was steel reinforced concrete column. The parameters investigated in this study were, configuration of the reinforcement for confinement, lap length of the hoop, volumetric ratio, and type of FRP reinforcement, glass versus carbon. Test results indicated that the behaviour of the CFRP and GFRP reinforced concrete columns were similar to the behaviour of the steel reinforced concrete columns up to peak load. A significant achievement in peak load of the CFRP reinforced concrete with increase of 3.4% for columns reinforced with helix and 2.16% increase for columns reinforced with hoops, as compared with GFRP reinforced concrete columns. Furthermore, the GFRP helix confined concrete columns gained 1.3% in strength and CFRP helix confined concrete columns gained 2.2% in strength, compared with that that gained by GFRP and CFRP hoops confined concrete columns, in terms of the influence of confinement configuration. The FRP reinforced concrete columns showed a ductile behaviour, and the strength decay rate at peak load was low compared to the steel reinforced concrete columns. Helix confinement compared to the hoops confinement was higher, but had insignificant effect on the ductility and confinement efficiency over strength capacity.
GFRP bars and helix or hoops had a higher confinement efficiency and ductility index compared to the CFRP bars and helix or hoops. Finally, the FRP and steel reinforced concrete columns failed in a ductile manner as the concrete cover spalled off gradually then buckling of the longitudinal bars took place followed by the rupture of the hoops or helix.

Tobbi et al. (2012) experimentally investigated the behaviour of concrete columns reinforced transversely and longitudinally with GFRP bars. The aim of this study was to examine the influence of the FRP bars as longitudinal and lateral reinforcement on the strength and strain capacities of the concrete members. In addition the effect of the tie configuration, tie spacing and spalling of the concrete cover were studied. The experimental work consisted of nine square specimens, with dimensions of 350 mm by 350 mm and 1400 mm height, five were GFRP reinforced concrete columns, two were steel reinforced columns and one was plain concrete column. In addition, four types of tie configuration were used in this study. Test results indicated that after the spalling of the concrete cover, a significant improvement in axial strength due to activation of the lateral confinement. In addition, smaller tie spacing leads to increased confinement efficiency. Furthermore, toughness, strength and ductility of the specimens were enhanced significantly by the GFRP tie configuration and spacing.

Tobbi et al. (2014) experimentally investigated the compressive performance of concrete columns reinforced with FRP or steel longitudinal bars and with FRP as transverse reinforcement. Twenty square concrete columns with side dimension of 350 mm and 1400 mm height were used in this study. Test variables included shape, spacing and materials of the transverse reinforcement, longitudinal reinforcement materials and ratio, and confining volumetric stiffness. Test results showed that acceptable strength and ductility behaviour were offered by the combination of the FRP transverse reinforcement and longitudinal steel
bars. The ultimate axial strain of the steel reinforced concrete specimens was 30% higher than the longitudinal FRP bars reinforced concrete specimens, with the same volumetric ratio. For the specimens with large spacing and low volumetric ratio, the performance of CFRP transverse reinforced columns was better than the GFRP transverse reinforced concrete columns. The failure mode of the exclusively FRP reinforced column was due to crushing or buckling of FRP longitudinal bars and rupture of the transverse reinforcement. For the columns reinforced with longitudinal steel bars, failure mode was due to buckling of the longitudinal steel bars. Explosive failure was observed in the specimens with the lower confinement volumetric stiffness, however for the specimens with the highest volumetric stiffness, failure mode was due to crushing of the GFRP bars followed by total concrete crushing.

Castro et al. (1995) experimentally investigated the compressive behaviour of FRP reinforced structural elements. A set of specimens with side dimension of 100 mm and with two different heights 800 mm and 1600 mm were used in his study. The compressive strength of the specimens was 20 MPa and 40 MPa. Test results showed that the columns with a height of 1600 mm had a significant FRP contribution due to buckling effect. Columns with low strength concrete were affected by FRP reinforcement. In addition, GFRP reinforced concrete columns had more compressive and buckling strength than plain concrete columns.

2.3 Behaviour of steel-tubes reinforced concrete columns

Steel tube-reinforced concrete columns have been studied experimentally by different researchers. Guo et al. (2010) experimentally investigated the compressive performance of the eccentrically loaded steel tube-reinforced concrete columns. In this study, thirteen specimens were used to study the changing of ultimate bearing capacity with the change of the eccentricity, and longitudinal reinforcement ratio. The division of the specimens was into
three groups according to different eccentricities, position coefficient and longitudinal reinforcement ratios used. In addition, this study obtained the deflection curve and load-vertical displacement curve of the specimens subjected eccentric compressive load. Test results showed that in the same test condition, with 2% steel tube ratio, the ultimate carrying capacity of steel tube-reinforced concrete columns was double the reinforced concrete columns. As the eccentricity increases, the steel tube-reinforced concrete columns showed similar failure behaviour to reinforced concrete columns. In addition, two types of failure characteristic were observed in steel tube reinforced concrete columns, damage due to small eccentricity and damage due to big eccentricity. Finally, for steel tube-reinforced concrete columns, with the increase of the longitudinal reinforcement ratio, the ultimate carrying capacity became larger. Figure 2.1 shows the cross section of the steel-tube reinforced concrete column.

![Figure 2.1 Steel-tube reinforced concrete column](image)

**Figure 2.1 Steel-tube reinforced concrete column**

Li et al. (2012) experimentally investigated the behaviour of concrete filled steel tube (CFST) under axial loading. Ten specimens with a length of 702 mm and diameter of 234 mm were used in this study. Eight 7.9 mm longitudinal bars were used for reinforcement of the specimens, and 6.4 mm plain bars used as transverse reinforcement with spacing of 70 mm. In addition, for core and outer concrete two different self-consolidating concrete were used to
test the effect of concrete strength on the behaviour of the concrete columns. Two different parameters were used for this study, ratio of steel tube and strength of concrete. Test results showed that the failure mode of the CFSTRC specimens was due to diagonal shear. The results showed that increased steel tube ratio led to increased ultimate strength of concrete columns. In addition, increased concrete strength for outer and core concrete led to increased ultimate strength of concrete columns. In addition, the core concrete strength has more influence on the ultimate strength of the CFSTRC columns than the strength of the outer concrete. Finally, after the test, the buckling in the steel tube was not obvious due to restrain of the outer reinforced concrete.

Ji et al. (2014) experimentally investigated the strength capacity of steel tube-reinforced concrete columns (ST-RC). Ten square concrete columns with side dimension of 300 mm and a height of 1100 mm reinforced with steel tube with diameter of 168 mm and thickness of 5.76 mm were cast for this study. Column specimens were subjected to axial force and lateral cyclic loading. Test variables were the applied axial load ratio to the columns and the amount of the transverse reinforcement. When the columns were subjected to identical axial compressive load, the ratio of the axial force for ST-RC column specimens was 30% lower than that of the RC columns, and the maximum bending moment for ST-RC was 22% higher than that of RC columns. The ratio of transverse reinforcement has a small influence on the lateral load carrying capacity of ST-RC columns; however, it significantly influences the deformation and energy dissipation capacity. Test results indicated that ST-RC have higher strength capacity than the reinforced concrete columns. In addition, the transverse reinforcement of the ST-RC has positive impact on ductility and deformation capacity. The failure mode of all the specimens was flexural mode.
Han et al. (2009) investigated the performance of concrete filled steel tube reinforced columns (CFSTRC) subjected to constant axial and cyclically increasing flexural loading. Nine specimens with three different cross sections were cast for this study. The first three specimens had square cross section reinforced with square CFST, the other three specimens had square cross and reinforced with circular CFST, and the last three had a circular cross section and reinforced with circular CFST. The authors investigated the effect of the level of axial load and cross section type on the strength, ductility, stiffness and energy dissipation. Test results showed that the ductility and dissipation of energy for CFSTRC columns were favourable even with high applied axial loads, but they decreased when the axial load increased. In general, the behaviour of the CFSTRC columns was ductile.

2.4. Behaviour of FRP reinforced concrete beams

Benmokrane et al. (1996) studied the flexural response of concrete beams reinforced with FRP reinforcing bars. The experimental program includes two series of beams with cross sections of 200 mm by 300 mm and 200 mm by 550 mm and length of 3300 mm. The comparison between the beams was for their cracking behaviour, load carrying capacity and failure modes, response of load-deflection, flexural rigidity and strain distribution. Test results indicated that at low load, the crack patterns for GFRP reinforced concrete beams was the same as that of concrete beams reinforced with steel, and at service load, concrete beams reinforced with GFRP had more and wider cracks than concrete beams reinforced with steel.

Masmoudi et al. (1998) investigated the flexural behaviour of concrete beams reinforced with deformed FRP rods subjected to static load. The experimental program consisted of four series, each series contained two identical beams with dimensions of 200 mm width, 300 mm height and 3300 mm length. The influence of the reinforcement ratio on the behaviour concrete beams was the main studied parameter. Test results showed that at low loading, the
average cracking spacing of beams reinforced with FRP bars was similar to that of steel reinforced beams. However, the cracking spacing for FRP reinforced beams at moderate and high loading was approximately 65% of the steel reinforced beams. In addition, results showed that the spacing of crack decreased as the load increased and the reinforcement ratio has a negligible effect on the cracking spacing. Also, the results showed that there was an increased in ultimate load with the increase of reinforcement ratio, but for concrete beams reinforced with GFRP, the increase was limited by compressive strain failure of the concrete. The mode of failure for over reinforced GFRP reinforced concrete beams was in two stage, stage one was due to crushing of the concrete and the second stage was due to fluctuation of the moment resistance strength.

Thériault and Benmokrane (1998) investigated experimentally the effects of the ratio of FRP reinforcement and strength of concrete on flexural behaviour of concrete beams reinforced with FRP rods tested under flexural loading. The experimental program consisted of twelve FRP bars reinforced concrete beams with dimensions of 130 mm width, 180 mm high and 1800 mm length. The main parameters for this study were reinforcement ratio and strength of concrete. In addition, the deflection, ultimate capacity and modes of failure were examined in this study. Test results indicated that there was an increase in crack width with the increase of concrete strength. In addition, the crack width increased with the decrease of the reinforcement ratio. Finally, concrete strength and reinforcement ratio had a negligible effect on the cracks spacing.

Ashour (2006) studied the flexural and shear capacities of concrete beams reinforced with GFRP bars. The experimental program consisted of twelve GFRP bars reinforced concrete beams with dimensions of 2100 mm length, 150 mm width and different beam depths. The
specimens were divided into two groups according to the compressive strength of concrete. The amount of the GFRP reinforcement and the depth of the beam were the main investigated parameters in this study. Test results observed a significant influence caused by the amount of the GFRP reinforcement on the beam’s flexural stiffness and deformations. In addition, test results indicated that there were two failure modes observed in this study, one of them is flexural failure and the other one is shear failure. The tensile rupture of GFRP bars under applied point load or in the mid span is called flexural failure, while, the diagonal cracks that appears within the beam shear span is called shear failure.

Badawi and Soudki (2009) studied the flexural strengthening of reinforced concrete (RC) beams with prestressed near-surface mounted (NSM) CFRP rods. The experimental program consisted of four RC beams with dimensions of 254 mm depth, 152 mm width and 3500 mm length. One of the beams was unstrengthened, to be used as reference specimen, one was prestressed CFRP bars strengthened concrete beam, while the other two were strengthened with a CFRP rods which prestressed up to 40% and 60% of the rod ultimate strength. However, the specimen that was strengthened with non-prestressed CFRP rods showed improvement in their flexural response, much improvement in flexural response was shown in specimens which were strengthened by prestressed CFRP rods. A significant increase in yield load and ultimate load of the strengthened specimens up to 90% compared to the reference beam. Furthermore, the cracking load for the strengthened beams with 40% - 60% prestressed CFRP rods increased up to 3 - 4 times compared to that of reference beam. The ultimate load for the beams reinforced with non-prestressed CFRP bars increased by up to 50% compared to the control specimen. The ultimate loads for the prestressed CFRP reinforced concrete beams increased by up to 79.2% and 60%, respectively. In addition, there were two failure modes were observed in the beams. One was for the control and non-
prestressed specimen due to crushing of the concrete at top fibre of the cross section, another one was for prestressed CFRP RC beams due to the rupture of the CFRP rod after yield of the tension steel. As the prestressing level of the CFRP rods increased, the ductility of the specimens decreased. The reduction in the ductility compared to control specimen was 30.6%, 47.2%, and 63.9% for 0%, 40%, and 60% prestressed CFRP bars, respectively.

2.5 FRP tube confined concrete columns

Li (2005) experimentally studied FRP- tube confined concrete columns (FRP/ECCs). The experimental program consisted of twenty-seven FRP/ECCs and they were divided into three groups with different concrete compressive strength. This study investigated the influence of strength of concrete on the ductility, bending strength, stress-strain behaviour and the strength of the interfacial bonding of FRP/ECCs. Test results indicated that the concrete strength had a significant influence on the structural behaviour of FRP/ECCs. On the other hand, high strength concrete has low efficiency of confinement and low interfacial bonding strength. For plain concrete specimens, the compressive strength increased significantly due to confinement of FRP tube. Results also showed that with the increase of the concrete strength, the strengthening efficiency of FRP tube decreases. Furthermore, the ductility and the time to activate the FRP tube were significantly affected by the concrete strength. It is also found that the interfacial bonding strength depends on many factors such as, dry shrinkage, Poisson’s ratio and chemical bond. Furthermore, the deflection of FRP/ECCs was influenced significantly by the core concrete strength.

Li (2006) experimentally studied the behaviour of concrete cylinders that confined by FRP. Two groups of FRP confined cylinders were used in this study, concrete cylinders that jacketed with FRP and concrete cylinders that encased with FRP tube. A total of twenty-four specimens of jacketed cylinders and six specimens of encased cylinders were used in this
study. The objective was to examine the structural behaviour of cylinders that were confined insufficiently, the effect of the concrete strength on the confinement efficiency and the effect of interfacial bonding on FRP tube confined concrete cylinders. Test results indicated that the confinement effectiveness rate of increase decreases non-linearly due to the increased confinement ratio. Furthermore, axial fibres FRP jackets had higher confinement effectiveness than other jackets. The cylinders with insufficient confinement showed no increase in strength. In addition, a higher ductility and compressive strength exhibited by encased cylinders with higher interfacial bonding strength.

Ozbakkaloglu and Oehlers (2008) investigated the behaviour of concrete filled square and rectangular FRP tubes subjected axial compression. The experimental program consisted of twenty-three columns with height of 600 mm, nine square specimens with side dimension of 200 mm and the remaining fourteen columns had rectangular cross section with dimensions of 150 mm by 300 mm. This study examined the influence of the tubes’ corner radius, thickness, sectional aspect ratio and strength of concrete on the axial behaviour of CFFTs. Test results indicated that for square and rectangular columns, a significant improvement in ductility was caused by the FRP confinement. In addition, for sufficiently high confinement effectiveness of FRP tube, axial load carrying capacity improved significantly for square and rectangular columns. The confinement effectiveness of the square columns is greater than that of rectangular concrete columns. The mode of failure of all the specimens was due to rupture of FRP tube near the corners. In addition, test results showed that for high confinement effectiveness, increasing the thickness of the FRP tubes has a significant effect on the ultimate strain and on ultimate strength if the tubes’ confinement effectiveness is high.
Park et al. (2011) experimentally investigated the mechanical performance of reinforced concrete filled FRP tube (RCFFT) by testing them under compressive and flexural loading. The experimental program included eighteen specimens of concrete filled FRP tube (CFFT) and six specimens of RCFFT. This study examined the confinement effect on the behaviour of CFFT and RCFFT in under compression. The test results showed that the compressive strength and the flexural ductility enhanced as the thickness of FRP tube increased. However, the compressive strength improved with the increase of confinement ratio, and it decreased with the increase of the ratio of height to diameter. Furthermore, the axial stiffness of core concrete improved due to the effect of FRP confinement.

Mohamed et al. (2014) investigated experimentally and theoretically the load capacity of axially loaded concrete-filled FRP tube columns (CFFT). The experimental program consisted of twenty-three small scale and medium scale CFFT tested under concentric load. The purpose of this study was to examine the influence of new filament wound GFRP tube on the compression behaviour of CFFT under axial loading. Influence of different parameters were examined in this study such as, confinement ratio of FRP, compressive strength of unconfined concrete, longitudinal steel bars presence, and the ratio of height to diameter. FRP tubes of different thicknesses and diameters were used in this study. Test result showed that increased thickness of FRP tubes affects significantly on the compressive strength and ductility of the CFFT cylinders.

Ozbakkaloglu (2013b) investigated the compressive behaviour FRP tube confined concrete columns. The results of ninety-two square, rectangular and circular concrete filled FRP tubes columns CFFTs were presented in this paper. The diameter of the circular specimens varied from 75 mm to 300 mm, while for the square and rectangular columns, the height used was
300 mm or 600 mm. This paper investigated the effect of strength of concrete, type and amount of materials for FRP tube, method of fabrication of FRP tube and size and shape of the CFFT's on the behaviour of CFFT's. Test results for circular concrete columns indicated that the behaviour of the CFFT's was significantly affected by mechanical properties of fibres in FRP tubes. On the other hand, manufacturing method of FRP tube has a small influence on the behaviour of CFFT's, but the effect of the specimen size was very small. For specimens that have square and rectangular cross section, the compressive behaviour was influenced significantly by the effectiveness of confining tube. For rectangular specimens, the compressive behaviour was influenced significantly by the radius of corner and section aspect ratio. The failure mode of the specimens was due to rupture of FRP tubes.

Ozbakkaloglu (2013a) investigated the behaviour of square and rectangle ultra-high strength concrete filled FRP UHSCFFT's tube under concentric loading. The experimental program consisted of twenty-four square and rectangular specimens with 300 mm height. This study investigated the influence of the amount of confinement, ratio of cross sectional aspect and radius of corner. Test results showed that a highly ductile behaviour can be exhibited by UHSCFFT's when they were sufficiently confined. As the radius corner increased and ratio of sectional aspect approached unity, the FRP tubes confinement effectiveness increased. However, the ultimate stress increased as the corner radius increased, it decreased with increased sectional aspect ratio. Furthermore, increased tube corner radius resulted in decreased ultimate strain. Also, the ultimate strain of UHSCFFT's affected significantly by FRP tube thickness. In addition, a highly ductile behaviour was exhibited by sufficiently square and rectangular confined UHSCFFT's. The mode of failure for all the specimens was due to FRP tube rupture.
Qasrawi et al. (2015) studied the dynamic behaviour of concrete filled FRP tube subjected to impact loading. The experimental program consisted of six specimens with length of 4000 mm and diameter of 200 mm, three of the specimens were CFFTs and three were steel reinforced concrete columns. The examined parameters in this study were presence of GFRP tube, ratio of internal steel reinforcement and input kinetic energy. Test results indicated that the flexural capacity and the maximum displacement increased by 112% because of confining the columns with FRP tube. In addition, the energy absorbing capacity increased by 467% and 1223% when the steel reinforcement ratio was 1.2% and 2.4%, respectively, compared to steel reinforced concrete columns. Furthermore, the ductility of the CFFTs was much higher than that of RC columns.

Ozbakkaloglu and Xie (2016) investigated the axial behaviour of concrete-filled fibre reinforced polymer FRP tube specimens (CFFTs). The experimental program consisted of thirty-six CFFTs. Eighteen of the specimens were cast by using Ordinary Portland Cement OPC, twelve specimens had a circular cross-section and six specimens were cast with square cross-section. The other eighteen specimens were cast by using fly ash-based Geopolymer Concrete GP, twelve specimens had a circular cross-section and six specimens had a square cross-section. The height of the specimens was 305 mm. The influence of the concrete type, FRP tube material, numbers of layers of FRP tube, and the cross-sectional shape of the specimens were investigated in this study. Test results indicated that concrete type has an effect on the axial stress-strain behaviour of the specimens. Furthermore, strength enhancement ratio for both Ordinary Portland Cement concrete filled FRP tube specimens OPCCFFTs and Geopolymer Concrete filled FRP tube specimens GPCFFTs were similar, but the axial strain enhancement ratio for GPCFFTs is lower than that for OPCCFFTs. The strength and strain ratios for OPCCFFTs that confined with different fibres were comparable.
On the other hand, a slight difference had been seen in the strength and strain ratios for GPCFFTs that were confined with different fibres. In addition, the strength and strain enhancement coefficients for OPCCFFTs and GPCFFTs decrease as the thickness of GFRP tube increase. GPCFFTs with circular cross section exhibited higher compressive strength, the ultimate axial strain and strength and strain enhancement coefficients than those with square cross section.

Xue and Gong (2016) experimentally studied the behaviour of steel reinforced concrete-filled GFRP tubular column under axial compression. The experimental program consisted of ten steel reinforced concrete-filled GFRP tubular columns and two concrete-filled GFRP tubular columns with length of 300 mm. Three types of GFRP tubes with inner diameter of 100 mm and thicknesses of 4 mm, 5 mm and 6 mm were used. Test results indicated that all the specimens failed by the rupture of the GFRP tube under hoop tension. Also, because of the presence of the concrete core and steel section, outward buckling can be observed in the specimens. In addition, after removing of the GFRP tube, crushing of concrete core and steel section buckling can be observed. The strength and deformation of new proposed columns are higher than that for concrete-filled FRP tubular columns because of presence of the section steel.

2.6 Steel tube confined concrete columns

Many studies have been conducted to examine the behaviour of steel tube confined concrete columns which their cross section can be seen in Figure 2.2.
De Nardin and El Debs (2007) studied experimentally the behaviour of axially loaded concrete-filled steel tubular columns. Six short column specimens with a length of 1200 mm were used in this study and tested under axial load. The specimens were with square, rectangular and circular cross sections, and two thicknesses of steel tubes were used for this study. The main objectives for this study were the load carrying capacity of concentrically loaded specimens and failure patterns up to and beyond the ultimate load. Test results indicated that the ductility increased for steel tube confined high strength concrete columns; however, the load carrying capacity remained the same. Furthermore, the square columns showed higher ductility than the rectangular specimens. In addition, for square concrete columns, the confinement efficiency was higher than that with rectangular cross section. On the other hand, the circular columns showed the highest ductility among the others. In general, the behaviour of the concentrically loaded specimens was due to concrete crushing and tubular steel section yielding. The behaviour of the specimens with circular cross section was in a brittle manner.

Liu et al. (2009) studied the behaviour and strength of reinforced-concrete confined by circular tube (CTRC) columns. The experimental program consisted of eighteen CTRC stub columns subjected to axial cyclic or monotonic compression. Five columns were tested under
a combination of axial compression and lateral cyclic load, four columns were CTRC columns and one was steel reinforced column. Test results showed that the flexural strength and displacement ductility for CTRC columns were higher than that for CRC hoop ties confined columns. In addition, the hysterias loops and energy dissipation ability for CTRC columns was higher than that for CRC columns. For CTRC columns, as the axial load and compressive strength of concrete increased, the flexural strength for the columns increased. On the other hand, the compressive strength of concrete and the axial load had a very small effect on the ductility. With the same parameters, circular tube reinforced concrete columns exhibited higher axial load strength than that of CFT columns.

Lai and Ho (2014a) experimentally examined the behaviour of uniaxial loaded concrete filled steel-tube columns CFST reinforced with external rings. The experimental work consisted of thirty-four normal strength and high strength CFST columns with different cross sections. Test results showed that the confined concrete columns had a higher stiffness and uniaxial strength than the unconfined concrete columns. In addition, the steel rings improved the interface bonding conditions and restricted the lateral dilation of CFST. Furthermore, the decreased spacing between the rings increases axial load carrying capacity and the increased axial strain increases the axial load. However, the failure mode for unconfined concrete columns was due to overall buckling of the steel tube, steel tube buckling was limited for the specimens confined with rings. For high strength CFST, because it has a brittle behaviour, their failure was due to fracture of the steel tube and rings while for normal strength CFST; there was no fracture in the steel tube and rings.

Ho et al. (2014) studied the uniaxial behaviour of confined high-strength concrete filled steel-tube (HSCFST) columns to examine the effectiveness of the external confinement in
eliminating the lateral dilation of the CFST columns. The enhancement of the confinement was examined by the achieved Poisson’s ratio, load-displacement curve, stiffness and axial load capacity. Strength and stiffness enhancement ratios were the main parameters adopted in this study. The experimental program consisted of twelve specimens with a diameter of 168 mm and a height of 330 mm, eleven were high strength CFST columns and one of the specimens was hollow steel tube specimen. Test results indicated that higher strength and stiffness were achieved by the specimens confined by external ties and rings. However, columns confined by rings had a higher effect on the strength, stiffness and deformability than the ties confined columns, ties confined HSCFST columns had higher axial strength than that of unconfined HSCFST columns. In addition, for the HSCFST columns, as the spacing of the rings increased, the axial strength increased. For the ring-confined HSCFST columns, the failure mode was due to local buckling of the steel tube between the rings, and for the tie confined columns the failure mode was due to local buckling of the steel tube at the ties opening near the mid height of the column. The improvement in the axial strength of HSCFST columns due to rings and ties was nearly 9% and 4%, respectively.

Lai and Ho (2014b) studied the effect of confinement for ring-confined concrete-filled steel tube (CFST) columns under axial load. Sixty-two CFST columns with different properties of material and cross sections were cast and subjected to uniaxial compression. Test results showed that the stiffness and axial load carrying capacity were enhanced by using rings, and the strength degradation decreased. In addition, the lateral deformation of the steel tube and concrete was limited by using rings. Furthermore, the load carrying capacity and stiffness can be improved by a maximum of 49% and 26% and an average of 8% and 6%, respectively, by installing additional external rings. CFST columns showed strain hardening behaviour due to adequate confinement. The failure for unconfined HSTs was due to local buckling of the steel
tube at the ends. On the other hand, the failure of confined HSTs columns was due to buckling of the steel tube between the rings.

2.7 Confinement models

A considerable number of confinement models have been conducted to examine the confinement effect on the behaviour of the concrete column. In this chapter, a summary of steel confined concrete models and FRP confined concrete models is presented.

2.7.1 Steel confinement models

The main use of transverse reinforcement is to avoid longitudinal steel bars buckling and shear failure. The steel reinforced concrete columns confinement consists of providing reinforcement in transverse direction in shape of helix, rectangular ties and circular hoops. Tests have proved that by using transverse confinement increases column’s strength and ductility. Many models have been proposed to study the steel confined concrete columns (Cusson and Paultre 1995, Imran and Pantazopoulou 1996, Lan and Gue 1997, Légeron and Paultre 2003, Madas and Elnashai 1992, Mander et al. 1988a, Mander et al. 1988b, Sfer et al. 2002). These models have been used widely in engineering practice. Furthermore, these two models have been used to adopt FRP confinement models. For Mander et al. (1988a) Model, the assumption of active confinement is the basement for this model. A constant confinement as well as uniform confinement is provided by this assumption to the core concrete. Furthermore, Mander et al. (1988a) model employs steel confinement stress-strain relationship of Popovics (1973) model, Figure 2.3. The compressive stress ($f'_c$) of Popovics model is given by

$$f_c = \frac{f'_{cc} \left( \frac{\varepsilon_c}{\varepsilon_{cc}} \right)^r}{r - 1 + \left( \frac{\varepsilon_c}{\varepsilon_{cc}} \right)^r} \quad (2.1)$$

This can be used for monotonic loading and low rate strain.
Where $\varepsilon_c$ is the concretes’ compressive strain of; $f_c$ is the axial concretes’ compressive stress; $f'_{cc}$ is the peak stress of confined concrete; compressive strain of confined concrete.

$$r = \frac{E_c}{E_c - E_{sec}}$$  \hspace{1cm} (2.2)

Where $E_c = 5000\sqrt{f'_{co}}$ MPa is the tangent modulus of elasticity for concrete; $E_{sec} = \frac{f'_{cc}}{\varepsilon_{cc}}$ is the second modulus of concrete.

For determining the axial compressive strength ($f'_{cc}$) and strain subjected to constant confinement, and by using active hydrostatic pressure confined concrete columns, Richart et al. (1928) explored Equations (2.3 & 2.4).

$$f'_{cc} = f'_{co} + 4.1 f_l$$  \hspace{1cm} (2.3)

$$\varepsilon_{cc} = \varepsilon_{co}[1 + 5 \left(\frac{f'_{cc}}{f'_{co}} - 1\right)]$$  \hspace{1cm} (2.4)

Where $f'_{co}$ = compressive strength of unconfined concrete; $\varepsilon_{co}$ = the axial train; $f_l$ = the confining pressure.

Figure 2.3 Unconfined and confined concrete Stress-strain curve

Mander et al. (1988a)
Mander et al. (1988a) model approached the effective confining pressure similarly to the method that examined by Sheikh and Uzumeri (1982). Figure 2.4 shows the estimated arching action that occurs in circular and rectangular transverse reinforcement and between the two levels of. Due to arching effect, the smallest confinement area is the middle area of the transverse reinforcement. Therefore, the coefficient of confinement effectiveness that is proposed by Mander et al. (1988a) for the actual confining pressure calculations is given by Equations (2.5a), (2.5b) and (2.5c).

For circular hoops;

\[ K_e = \frac{(1 - \frac{S^s}{2d_s})^2}{1 - \rho_{cc}} \]  \hspace{1cm} (2.5a)

For circular helices;

\[ K_e = \frac{1 - \frac{S^s}{2d_s}}{1 - \rho_{cc}} \]  \hspace{1cm} (2.5b)

For rectangular hoops;

\[ K_e = \frac{[1 - \sum_{i=1}^{n} (W'_i)^2/(6b_c d_c)](1 - \frac{S'}{2b_c})(1 - \frac{S'}{2d_c})}{1 - \rho_{cc}} \]  \hspace{1cm} (2.5c)
Figure 2.4 Concrete area that effectively confined by circular hoop reinforcement

Mander et al. (1988a)

Where $\rho_{cc}$ is the ratio between longitudinal reinforcement area and core of section area; $W_i' = l_{th}$ the $i_{th}$ clear distance between two neighbours longitudinal bars; $b_c$ and $d_c$ = dimensions of core to centre lines of confines hoop in the directions of x and y, respectively; where $b_c>d_c$.

The actual confining pressure can be calculated by

$$f'_l = f_l K_e$$ (2.6)

Where $f_l$ = the actual confining pressure which is found by the consideration of the half body confined by circular hoop or helix. Based on the assumption of the active confinement, the transverse steel at yield developed a uniform hoop tension. The lateral core concrete stress is derived from the equilibrium of the forces.

$$f_l = \frac{2f_{sy}A_{sp}}{Sd_c}$$ (2.7)

Where $s$ is the spacing of the circular hoop or helix centre to centre; $d_s$ is the core concrete diameter and can be measured as helix or circular hoop centre to centre diameter; $f_{sy}$ = yield stress of the steel and $A_{sp}$ = cross section area of the transverse reinforcement. The compressive strength which is adopted by Mander et al. (1988a) is given by Equation 2.8. This equation is depends on five parameters surface of multiaxial failure. The terms of two lateral confining stresses presented the solution of the failure criterion. When there is a
triaxial compression with equal effective transverse steel lateral confining pressure effects on concrete core, the compressive strength for confined concrete can be calculated by Equation 2.8.

\[ f'_{cc} = f'_{co} \left( -1.254 + 2.254 \sqrt{1 + \frac{7.94f'_{l}}{f'_{co}}} - 2 \frac{f'_{l}}{f'_{co}} \right) \]  

(2.8)

Priestley (1996) developed the ultimate compressive strain or axial compressive strain at rupture, which can be defined as the first fracture of transverse reinforcement.

\[ \varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{sy} \varepsilon_{cu}}{f'_{cc}} \]  

(2.9)

Where \( \varepsilon_{cu} \) = the steel strain at maximum tensile stress; and \( \rho_s \) = the volumetric ratio of confinement reinforcement to the confined core concrete;

2.7.2 FRP confinement models

The steel confinement models cannot be used as models for FRP materials confined concrete. This is because FRP has different characteristic from that of steel (Mirmiran and Shahawy 1996, Samaan et al. 1998, Xiao and Wu 2000). Figure 2.5 shows the difference in stress – strain behaviour between steel and FRP confined concrete. It can be seen that before peak stress \( f'_{cc} \), the steel confined concrete observed a softening, followed by a gradual descending part. The peak stress \( f'_{cc} \) at failure is higher than stress \( f'_{cu} \); on the other hand, sharp softening and transition zone is experienced by FRP-confined concrete, before stress reaching its unconfined strength \( f'_{co} \), it is followed by a stabilized tangent at a constant rate until the achievement of the ultimate strength \( f'_{cu} \).
Figure 2.5 FRP and steel confined concrete Stress –strain curve

(Samaan et al. 1998)

There have been a considerable number of studies conducted on concrete confined by FRP materials (De Lorenzis and Tepfers 2003, Lam and Teng 2002, Lam and Teng 2003a, Lam and Teng 2003b, Lam and Teng 2004, Mirmiran and Shahawy 1997, Pessiki et al. 2001, Rochette and Labossière 2000, Spoelstra and Monti 1999), and many other models. (Ozbakkaloglu et al. 2013b) presented a comprehensive review study developed to predict the axial stress-strain behaviour of FRP confined concrete in circular cross-sections. In this study, eighty-eight models from 1991 to 2011 were reviewed and assessed. A reliable data based from 202 experimental studies which covered 2038 test results were used in the assessment of the models. The reviewed models were classified into two main categories (1) design oriented models and (2) analysis oriented models. Fifty-nine models out of the eighty-eight models were classified as design oriented models; thirteen models were classified as analysis oriented models, while sixteen models were classified into models based on other approaches. The results from this study clarify that the design oriented models perform better than analysis oriented models. In addition, this study revealed that the models of (Lam and Teng 2003a) and (Tamuzs et al. 2006) are the most accurate design oriented models for the
prediction of the ultimate stress and strain enhancement ratio of FRP confined concrete columns.

2.7.2.1 Design oriented model

The proposition of the most design oriented models is based on stress strain curves of different tests of the circular concrete specimens that confined by FRP (De Lorenzis and Tepfers 2003, Lam and Teng 2003a, Ozbakkaloglu et al. 2013a, Samaan et al. 1998, Tamuzs et al. 2006, Wang et al. 2015, Xiao and Wu 2000). The models that were defined as the most precise models are Lam and Teng (2003a), Lam and Teng (2003b) and model of Samaan et al. (1998). Samaan et al. (1998) model is the improved form of Richard and Abbott (1975) four parameters relationship in order to define the elastic plastic performance of the structural system, as can be seen in Figure 2.6.

![Four parameter stress-strain curve](image)

Figure 2.6 Four parameter stress-strain curve (Richard and Abbott 1975)

\[
f_c = \frac{(E_1 - E_2)E_c}{\left(1 + \left(\frac{E_1 - E_2}{f_o}\right)E_c\right)^n} + E_2E_c \tag{2.10}
\]

Where \(f_c\) is confined concrete stress and \(E_c\) is the strain of confined concrete, \(E_1\) is the first slope of the curve \(E_2\) is the second slope of the curve; \(f_o\) = reference plastic stress; and \(n\) is transition controller parameter that controls the transition between the first to the second
portion of the curve. Test results of 30 concrete filled FRP tube were used by Samaan et al. (1998) to calibrate the model.

In addition, for FRP tube confined concrete Lam and Teng (2003a) design oriented stress-strain model has been used widely in practical application for FRP confined concrete columns. The ACI 440.2R (2008) adopted the model of Lam and Teng (2003a) for FRP confined concrete. In this model the stress-strain curve of FRP confined concrete is described by the parabolic first portion with the linear second portion. The expression of stress-strain model that was proposed by (Lam and Teng 2003a) are as follow:

\[
\sigma_c = E_c \varepsilon_c - \frac{(E_c - E_2)^2}{4f_o} \varepsilon_c^2 \quad 0 \leq \varepsilon_c \leq \varepsilon_t \quad (2.11)
\]

\[
\sigma_c = f_o + E_2 \varepsilon_c \quad \varepsilon_t \leq \varepsilon_c \leq \varepsilon_{cu} \quad (2.12)
\]

Where \(\sigma_c\) is the axial stress; \(\varepsilon_c\) is the axial strain; \(E_c\) is the elastic modulus of unconfined concrete; \(E_2\) is the slope of the linear second portion of the stress-strain curve; \(f_o\) is the intercept of the stress axis by strain curve; and \(\varepsilon_{cu}\) is the ultimate axial strain of confined concrete. In the stress-strain curve, the first parabolic portion meets the second linear portion at \(\varepsilon_t\). The \(\varepsilon_t\) can be calculated by using the following expression:

\[
\varepsilon_t = \frac{2f_o}{(E_c - E_2)} \quad (2.13)
\]

The second linear portion slope \(E_2\) is given by Equation 2.14

\[
E_2 = \frac{f'_{cc} - f_o}{\varepsilon_{cu}} \quad (2.14)
\]

Where \(f'_{cc}\) is the confined concrete compressive strength; \(f_o\) value is considered to be equal to the unconfined concrete compressive strength \(f'_{co}\). The ultimate compressive and ultimate axial strain of FRP confined concrete can be calculated by using the following equations:
\[
\frac{f_{cc}'}{f_{co}'} = \begin{cases} 
1 - 3.3 f_{tu,a} \frac{f_{tu,a}}{f_{co}} & , \quad \frac{f_{tu,a}}{f_{co}} \geq 0.07 \\
1 & \quad \frac{f_{tu,a}}{f_{co}} < 0.07
\end{cases}
\] 

(2.15)

\[
\frac{\varepsilon_{cu}}{\varepsilon_{co}} = 1.75 + 12 \left( \frac{f_{tu,a}}{f_{co}} \right) \left( \frac{\varepsilon_{rup}}{\varepsilon_{co}} \right) 0.45
\]

(2.16)

Where \( f_{tu,a} \) is the actual lateral confining pressure at \( \varepsilon_{cu} \); and \( \varepsilon_{co} \) is the axial strain of unconfined concrete at peak strength \( f_{co}' \).

2.7.2.2 Analysis oriented model

The analysis oriented model development is based on incremental interactive numerical approach, (Fam and Rizkalla 2001, Mirmiran and Shahawy 1996, Spoelstra and Monti 1999).

The FRP jacket hoop strain, the corresponding axial strain, relation FRP confinement pressure and the concrete are the basic independent parameters for this type of model. These models are not suitable for manual design because of their complex nature. However, these models are more effective in the design process by using computer analysis.

2.8 Summary

In conclusion, FRP bars reinforced concrete columns and beams, steel tube and FRP tube confined concrete columns, and steel tube reinforced concrete columns had been studied to improve the load carrying capacity and ductility and the structural behaviour of the concrete structures. Nevertheless, there are many disadvantages with these techniques. Therefore, FRP tube reinforced concrete columns have been studied to avoid the limitations of these techniques. Research proved that FRP tube RC columns have an excellent load carrying capacity in addition to their high corrosion resistance. In addition, the concrete cover can be used to protect the FRP tube from external effects and cover spalling can be used as indication before failure.
Chapter 3 Experimental Program

3.1 Introduction

The behaviour of GFRP tube-reinforced concrete columns under concentric, eccentric and flexural loading was examined experimentally. The experiments were carried out at the University of Wollongong (High Bay civil engineering laboratory). The preparation, casting, curing and testing of the specimens were done at the laboratory. The materials were provided by local suppliers except the CFRP sheets which were imported from China. The following parts in this chapter present more details about the experimental program.

3.2 Design of specimens

The experimental program consisted of sixteen circular concrete specimens. The diameter of the specimens was 240 mm and height 800 mm. Four of the sixteen specimens were reinforced with steel bars and helix. The steel reinforced concrete specimens were used as reference group. Six deformed bars of 12 mm (N12) diameter and nominal tensile strength of 500 MPa were used as longitudinal reinforcement and R6 mm plain bars (6-mm plain bars with nominal tensile strength of 250 MPa) tied at 50 mm spacing were used as helix. The other twelve concrete specimens were reinforced with GFRP tube which replaces the longitudinal reinforcement and transverse confinement. The GFRP tubes had an outer diameter of 183 mm, thickness of 8 mm and height of 760 mm. The concrete specimens were cast with normal strength concrete with compressive strength of 32 MPa. The concrete cover of the specimens was 28 mm side cover and 20 mm for top and bottom cover. CFRP sheet with 100 mm width and four layers was used to wrap the ends of the specimens before testing to prevent premature failure at the end of the specimens.
3.3 Test configuration

The sixteen reinforced columns were divided into four groups; each group consists of four specimens with diameter of 200 mm and height of 800 mm. Specimens in the first group (Group REF) which is the reference group, were reinforced with longitudinal steel bars and steel helixes. Specimens in the second group (Group ST) were reinforced with intact glass FRP tubes. Specimens in the third group (Group ST-G) were also reinforced with intact glass FRP tubes. In addition, two layers of polymer grid were embedded into the concrete cover to reduce the cover spalling. Specimens in the fourth group (Group PT) were reinforced with perforated glass FRP tubes to integrate concrete cover with concrete core. Three vertical lines of holes with diameter of 15 mm and 75 mm vertical holes spacing were made in each tube with equal distance between each hole. In each group, the first specimen was tested under concentric loading; the second and third specimens were tested under eccentric loading with 25 mm and 50 mm eccentricities, respectively. The fourth specimen was tested under flexural loading to evaluate its bending behaviour. The specimens were labelled according to group name and loading condition of the tested specimen, where the number refers to loading eccentricity and (F) refers to flexural test. For example REF-0 represents the specimen in Group REF (reference group) tested under concentric load. Another example, ST-F represents specimen in Group ST (solid tube reinforced concrete specimens) tested under flexural loading. The details of the specimens are shown in Table 3.1.
### Table 3.1 Details of the test specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Reinforcement type</th>
<th>Test modes</th>
</tr>
</thead>
<tbody>
<tr>
<td>REF-0</td>
<td>Steel reinforcement</td>
<td>Concentric loading</td>
</tr>
<tr>
<td>REF-25</td>
<td></td>
<td>Eccentric loading, e=25 mm</td>
</tr>
<tr>
<td>REF-50</td>
<td></td>
<td>Eccentric loading, e=50 mm</td>
</tr>
<tr>
<td>REF-F</td>
<td></td>
<td>Flexural loading</td>
</tr>
<tr>
<td>PT-0</td>
<td>Perforated GFRP tube</td>
<td>Concentric loading</td>
</tr>
<tr>
<td>PT-25</td>
<td></td>
<td>Eccentric loading, e=25 mm</td>
</tr>
<tr>
<td>PT-50</td>
<td></td>
<td>Eccentric loading, e=50 mm</td>
</tr>
<tr>
<td>PT-F</td>
<td></td>
<td>Flexural loading</td>
</tr>
<tr>
<td>ST-0</td>
<td>Solid GFRP tube</td>
<td>Concentric loading</td>
</tr>
<tr>
<td>ST-25</td>
<td></td>
<td>Eccentric loading, e=25 mm</td>
</tr>
<tr>
<td>ST-50</td>
<td></td>
<td>Eccentric loading, e=50 mm</td>
</tr>
<tr>
<td>ST-F</td>
<td></td>
<td>Flexural loading</td>
</tr>
<tr>
<td>ST-G-0</td>
<td>Solid GFRP tube with grid mesh</td>
<td>Concentric loading</td>
</tr>
<tr>
<td>ST-G25</td>
<td></td>
<td>Eccentric loading, e=25 mm</td>
</tr>
<tr>
<td>ST-G-50</td>
<td></td>
<td>Eccentric loading, e=50 mm</td>
</tr>
<tr>
<td>ST-G-F</td>
<td></td>
<td>Flexural loading</td>
</tr>
</tbody>
</table>

### 3.4 Formwork set up

The formwork for the experimental program consisted of two parts, one for concrete specimens and the other for the whole specimens. The formwork for the specimens was made from PVC pipes with an inner diameter of 240 mm. Those pipes were supplied by a local supplier, and they were cut to a length of 800 mm to form the formwork for the specimens, as shown in Figures 3.3.
The pipes for the formwork were attached to the timber base as shown in Figure 3.1. In addition, silicon sealant was used to close the spaces between the plywood base and the PVC pipes to prevent the seepage of concrete water from the contact area.

3.5 Preparation of the specimens

3.5.1 Steel reinforced concrete columns

Deformed bars were used as longitudinal reinforcement and they were supplied by local supplier. These bars were cut into small pieces with a length of 760 mm. In addition, plain bars were used as helix; they were manufactured and rounded by a local company. The longitudinal steel bars and helix were connected to each other by using steel wires. Six longitudinal steel bars were used in each specimen. After preparation of the steel cages and to make sure that the specimens will have enough concrete cover after pouring of concrete, 20 mm and 28 mm length of steel bars were welded on the bottom and the sides of the steel cages, respectively. After welding of the steel bars, the steel cages were held inside the PVC pipe, as shown in Figure 3.2.
3.5.2 GFRP tube reinforced concrete columns

Twelve GFRP tubes were used in the experimental program, four of them were perforated tubes to perform integration between the concrete cover with the core concrete. Strain gauges were attached for the measurement of axial deformation. For the GFRP tubes which were tested under concentric and eccentric loading, four strain gauges were attached, two in each side, one horizontally and the other attached vertically. For the GFRP tubes which were tested under flexural loading, six strain gauges were attached. Two strain gauges were attached at the centre point of four holes, two at centre point of two horizontal holes and the last two were attached at the centre point of two vertical holes. At all three locations of strain gauges, one strain gauge was attached in the horizontal direction and one in the vertical direction. Short steel wires were used to provide sufficient side cover of 28 mm between GFRP tube and PVC pipes. In addition, for Group ST-G, plastic grid mesh was used in four specimens between the GFRP tube and plastic formwork. The PVC pipes were fixed in their location inside the wooden formwork. Then, steel cages and the GFRP tubes were held inside the PVC pipes. Figure 3.3 shows the details of GFRP tubes.
Figure 3.3 GFRP tubes in different configurations

3.6 Pouring of concrete

A normal strength concrete with a compressive strength of 32 MPa which was supplied by a local supplier was used for casting the specimens. A slump test was used to ensure that the concrete has a workability. The concrete was poured inside the formwork carefully with three layers for each specimen. Hand held vibrator was used to eliminate air bubbles inside the concrete. In addition, nine small cylinders made of steel with dimensions of 100 mm in diameter and 200 mm in height were poured for Preliminary tests. The cylinders were cast with two layers; each layer compacted by steel rod with 25 strike. Figure 3.4 shows stages of concrete pouring.
Figure 3.4 Stages of concrete pouring

3.7 Concrete curing

After one day of the concrete pouring, curing started and continued at 28 days. Moist hessian was used for curing in room temperature.

3.8 Preliminary tests

Tests results of the concrete cylinders showed that the average compressive strength at 28 days was 35 MPa. On the other hand, the tensile strength of the steel bars used in these specimens was examined according to AS1391 (2007). Three samples of N12 deformed bars and R10 plain bars with length of 300 mm were tested. The average tensile strength from the test was 400 MPa and 440 MPa for the R12 and N12, respectively. In addition, tensile properties of the polymer grid were determined by testing polymer grid strand using Instron.
8033 machine. Each end of the polymer grid strand was embedded in steel clamps. The two steel plates were then tightened towards each other in order to fix the polymer grid. The total length of the polymer grid strand was 158 mm with a free length of 102 mm. The displacement controlled test was carried out at a rate of 3 mm/min. A linear elastic behaviour was observed, and the average tensile strength of the polymer grid was 484 MPa with an elastic modulus of 5 GPa. The GFRP tubes were tested under compression in accordance with GB/T 5350 (2005). Before testing, the tube was placed onto the bottom loading plate to check whether there was any misalignment between the tube end and the bottom loading plate. If a slight misalignment was observed, the tube end was slightly smoothed using a belt sander until the misalignment was removed. The test was conducted at a rate of 0.3 mm/min. The average axial compressive strength of GFRP tube was 416 MPa with a corresponding axial strain of 0.0145. The elastic modulus was 28.7 GPa. Due to the limitations of the experimental setup, the hoop tensile properties of the GFRP tubes could not be experimentally obtained. Therefore, the hoop tensile properties of the GFRP tubes provided by the manufacturers were used for further analysis. The hoop ultimate tensile strength of GFRP tube and the hoop modulus of elasticity provided by the manufacturer are 50 MPa and 10 GPa, respectively. Figure 3.6 show the stress-strain curve for GFRP tube. In addition, no test had been conducted for the perforated GFRP tube.

![Stress-Strain diagram for steel bars](image)

**Figure 3.5 Stress-Strain diagram for steel bars**
3.9 Preparations of the specimens for the test

After 28 days of curing, the specimens were taken out from the formwork and prepared for the test. In order to prevent premature failure at the ends of the columns, CFRP wrapping with four layers was used to wrap the column specimens’ ends. The CFRP was bond to the column specimens with adhesive by using a wet lay-up method. The mixture of the adhesive was made by mixing an epoxy resin and slow hardener. The CFRP sheet that was used for the wrapping had a width of 100 mm. The method of wrapping was done by spreading of the adhesive on the surface of the column specimens. After that the 100 mm wide CFRP sheets were wrapped around the column specimens in the hoop direction. The adhesive was spread on the surface of the first layer and then the second layer wrapped around the column, and the same procedure for the third and fourth layers. After two days of wrapping, the specimens were ready for testing in the compression machine. Figure 3.7 shows the specimens before and after wrapping.
3.10 **Configuration of the loading system**

For loading tests, the Dension testing machine with 5000 kN compression capacity was used, as can be seen in Figure 3.8.
For eccentric loading tests, a special loading system was used. The loading system consists of two loading heads and two knife edges made from high strength steel. Each loading head has three grooves which located at 15 mm, 25 mm and 50 mm from the centre of the loading head. In order to test the specimens under 25 mm or 50 mm eccentricity, the knife edge was put at the groove of 25 mm and 50 mm, respectively, as can be seen in Figure 3.9. A four point loading system was used for flexural tests. The system consists of two loading segments one at the top and the other at the bottom of the specimen. For the upper segment, the distance between the two edges is 230 mm c/c and for the bottom segment, the distance is 700 mm c/c, as can be seen in Figure 3.10.

Figure 3.9 Loading heads and knife edge
Each loading segment has two rounded edges for transferring load from the loading machine to the specimen. Before the placement of the column specimens in the machine, both of the specimens’ ends were covered with high strength plaster in order to make sure that the load distributed equally on the specimens. The mixture of high strength plaster was poured into the bottom loading head then the column specimen was fixed inside the loading head. After that, a plaster was poured above the upper end of the column specimens and then the loading head was fixed above the upper end. The screws of the loading caps were tightened and the column specimens were lifted to the testing machine.

A small loading of 30 kN was applied to make sure that the loading heads compressed totally. After 30 minutes of curing for each end, the specimen was ready for the test. A laser triangulation displacement sensor was used for eccentric loading and flexural loading tests. The lateral displacement was measured by putting it at the mid height of the column. For the measurement of midspan deflection in flexural test, the laser triangulation displacement sensor was fixed in the middle of the loading plate. A laser linear variable differential transformer (LVDT) was used to measure the axial displacement and loading during the test. This LVDT is located at the bottom load platform of the test machine. The data from the laser triangulation displacement sensor and LVDTs were recorded every 2 seconds by connecting them to a data-logger.
3.11 Summary

An experimental program was undertaken to examine the behaviour of GFRP tube-reinforced concrete columns under concentric, eccentric and flexural loading. Sixteen concrete specimens were used in the experimental program including four steel reinforced specimens which were used as reference. The details of the specimens’ preparation, material testing and concrete pouring were described above. In addition, a brief description of the machine used for testing was introduced. The following chapter presents an analysis of the experimental results.
4.1 Introduction

This chapter presents the analysis of the results from the experimental program including a description of the failure mechanism of the specimens under concentric, eccentric and flexural loading. An evaluation of the axial load-bending moment (P-M) interaction diagram and calculation of the ductility are also presented in this chapter.

4.2 Specimens under concentric loading

The experimental program of this study consisted of sixteen specimens divided into four groups; one specimen from each group was tested under concentric loading. The failure mechanism was due to buckling of longitudinal steel bars for steel reinforced specimens (SRCs) and rupture of GFRP tube for FTRC specimens.

4.2.1 Mechanism of the failure

The failure mechanism of Specimen REF-0 was due to cover spalling and buckling of the longitudinal steel bars. Spalling of the concrete cover occurred when the specimen achieved its yield load. After the complete spalling of concrete cover, the core concrete expanded laterally which caused the buckling of the longitudinal steel bars.

For Specimen PT-0, the failure was due to rupture of the GFRP tube in the hoop direction. During the test, cracks appeared in the concrete cover after the cover started to spall off and the load started to decrease steadily. Then the load increased again and reached its maximum value due to activation of the core concrete. After a while, the load dropped suddenly due to the rupture of the GFRP tube which followed by a loud sound. The early failure of the specimen PT-0 was due to the premature failure at the end of the specimen. The premature failure problem was solved for the other specimens by wrapping the ends with four layers of CFRP sheets. For Specimen ST-0, the failure mechanism was the same as Specimen PT-0 but
it achieved maximum load higher than maximum load for Specimen PT-0 because there was no premature failure. The concrete cover totally spalled off and the GFRP tube cracked due to low hoop strain and expansion of the core concrete. Specimen ST-G-0 had the same failure mechanism as Specimen PT-0 and Specimen ST-0 but the concrete cover cracked without spalling because of using polymer grid inside the cover. Figure 4.1 shows the failure mechanisms of the specimens tested under concentric loading.

Figure 4.1 Failure mechanisms of the specimens under concentric loading
4.2.2 Test results

Test results showed that for concentric loaded specimens, Specimen ST-0 achieved the highest maximum load, which was 24.5% higher than that for reference Specimen REF-0. In addition, Specimen ST-G-0 achieved maximum load higher than that for REF-0 of about 24.4%. On the other hand, Specimen PT-0 achieved a maximum load lower than that for REF-0 of about 4.85%. Low maximum load was due to premature failure at the ends of the specimen. Specimen REF-0 which was tested under concentric loading had the highest axial deformation of 40.41 mm. According to (Pessiki and Pieroni 1997), the yield load is defined as the load that is equivalent to an initial displacement calculated corresponding to the intersection of the best fit line to the linear portion of the diagram with the maximum load.

Table 4.1 Test results of the specimens tested under concentric loading

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield load (kN)</th>
<th>Corresponding axial deformation (mm)</th>
<th>Maximum load (kN)</th>
<th>Corresponding axial deformation (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>REF-0</td>
<td>1486.5</td>
<td>2</td>
<td>1486.5</td>
<td>3</td>
</tr>
<tr>
<td>PT-0</td>
<td>1119</td>
<td>1.95</td>
<td>1414.6</td>
<td>5.3</td>
</tr>
<tr>
<td>ST-0</td>
<td>1515</td>
<td>2.4</td>
<td>1850.1</td>
<td>6.2</td>
</tr>
<tr>
<td>ST-G-0</td>
<td>1442.6</td>
<td>2.1</td>
<td>1849.1</td>
<td>6.04</td>
</tr>
</tbody>
</table>

4.2.3 Load-deformation diagrams

Figure 4.2 shows axial load-deformation diagrams for the specimens tested under concentric load. It can be seen that all the specimens had the same behaviour at the initial stage before the yield load. All the specimens mostly had the same yield load except Specimen PT-0 which had a yield load less than yield load for other specimens. This is because of the premature failure at the ends of Specimen PT-0 which caused in an early failure. In addition,
the axial deformation for Specimen REF-0 is much higher than the axial deformation in other specimens, which means that Specimen REF-0 has higher deformation capacity than other specimens. On the other hand, the maximum loads for Specimens ST-0 and ST-G-0 are much higher than REF-0. This may be due to the confinement efficiency of GFRP tube which increased the load carrying capacity for FTRCs. For Specimen PT-0, the maximum load is less than Specimen ST-0 and specimen REF-0 because of the premature failure at the ends. It can be recognised that FTRCs failed suddenly due to the rapture of GFRP tube.

![Figure 4.2 Load-deflection diagrams of concentrically loaded specimens](image)

**Figure 4.2** Load-deflection diagrams of concentrically loaded specimens
4.3 Specimens tested under eccentric loading

In each group, two specimens were tested under 25 mm and 50 mm eccentric loading. The test was stopped after longitudinal bars buckling of the steel reinforced specimens (SRCs) and rupture of the GFRP tube for FTRCs.

4.3.1 Mechanism of the failure

Specimens REF-25 and REF-50 failed by the buckling of the longitudinal steel bars and crushing of the concrete at the compression region. For Specimen REF-25, with the increased load, cracks started to appear at the tension side and then the concrete cover started to spall off but the specimen was able to carry higher load. After that, the longitudinal bars buckled at the compression region. For Specimen REF-50, it had nearly the same failure mechanism as Specimen REF-25 with different maximum load and axial deformation.

Specimens PT-25 and PT-50, the failure was due to rupture of the GFRP tube in the hoop direction. For Specimen PT-25, the failure mechanism started by cracks appearing on the cover at the tension region. The cracks continued to grow with the increased load. The concrete cover took longer to spall than the cover of Specimen ST-25. This is because, the holes in the GFRP tube allowed for the concrete cover to form a mechanical interlock with the core concrete. A sudden failure happened in the specimen due to rupture of the GFRP tube in the hoop direction. The main cracks in the GFRP tube were along or near the holes. For Specimen PT-50, the failure mechanism was nearly the same as Specimen PT-50 with different maximum load and deformations.

Specimens ST-25 and ST-50 failed by the rupture of the GFRP tube in the hoop direction. For specimen ST-25, cracks started to appear at the tension region and grew with the increase of the load, the concrete started to spall off at the compression region. After reaching its maximum load, the GFRP tube ruptured suddenly with a loud sound. The GFRP tube
ruptured due to low hoop strain and expansion of the core concrete inside the tube. Specimen ST-50 had approximately the same failure mechanism as Specimen ST-25 with different maximum load and deformations.

Specimens ST-G-25 and ST-G-50 failed by the rupture of the GFRP tube. For Specimen ST-G-25, cracks appeared in the concrete cover and grew with the increased load. The concrete cover did not spall off completely because a polymer grid was used inside the cover. As a result, the majority of the cover concrete cracked but without spalling. Sudden failure happened due to reaching the maximum load; the failure was because of the rupture of the GFRP tube. The cracks shape of the GFRP tube was unclear because of the unspalled concrete cover. Specimen ST-G-50 had the same failure mechanism as Specimen ST-G-25 with different maximum load. The effect of the polymer grid on the maximum load was insignificant; the effect was only on preventing and delaying the concrete cover spalling. Figures 4.3 and 4.4 show the failure mechanism of the specimens under eccentric loading.

4.3.2 Test results

Table 4.2 shows the results of the specimens under eccentric loading. For the specimens that were tested under 25 mm eccentric loading, Specimen ST-G-25 achieved the highest maximum load with an increase of 58.2% compared to Specimen REF-25, followed by Specimen ST-25 with an increase of 50% and then Specimen PT-25 with an increase of 42.2% compared to Specimen REF-25.

For the specimens that were tested under 50 mm eccentric loading, Specimen ST-G-50 achieved the highest maximum load with an increase of 50.3% compared to Specimen REF-50, followed by Specimen PT-50 with an increase of 49% and then Specimen ST-50 with an increase of 31.5% compared to Specimen REF-50.
4.3.3 Load-deformation diagram

Table 4.2 Test results of the specimens tested under eccentric loading

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield load (kN)</th>
<th>Corresponding axial deformation (mm)</th>
<th>Maximum load (kN)</th>
<th>Corresponding axial deformation (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>REF-25</td>
<td>985</td>
<td>2.5</td>
<td>985.3</td>
<td>2.5</td>
</tr>
<tr>
<td>PT-25</td>
<td>1000</td>
<td>3.5</td>
<td>1400.4</td>
<td>6.1</td>
</tr>
<tr>
<td>ST-25</td>
<td>1091</td>
<td>2.8</td>
<td>1476.9</td>
<td>6.6</td>
</tr>
<tr>
<td>ST-G-25</td>
<td>1091</td>
<td>2.9</td>
<td>1558.5</td>
<td>7.1</td>
</tr>
<tr>
<td>REF-50</td>
<td>696</td>
<td>2.0</td>
<td>696.1</td>
<td>2.5</td>
</tr>
<tr>
<td>PT-50</td>
<td>740</td>
<td>2.8</td>
<td>1037.7</td>
<td>9.5</td>
</tr>
<tr>
<td>ST-50</td>
<td>733</td>
<td>2.5</td>
<td>915.3</td>
<td>9.6</td>
</tr>
<tr>
<td>ST-G-50</td>
<td>719</td>
<td>3.1</td>
<td>1046.0</td>
<td>8.8</td>
</tr>
</tbody>
</table>

Figure 4.5 shows the load deformation diagram of the specimens tested under eccentric load with 25 mm eccentricity. It can be seen that all the specimens had the same behaviour before the yield point. In addition, all the specimens had mostly the same yield load. After the yield load, load increased again for all specimens because of activation of GFRP tube confined concrete core except for Specimen REF-25. Specimen ST-G-25 had the highest maximum load compared to the other specimens. This may because of using polymer grid in the concrete cover which has a small influence on maximum load of the whole specimen. Specimen PT-25 achieved the lowest maximum and this may be due to the holes on the GFRP tube which decrease the confinement efficiency of the GFRP tube. Specimen REF-25 had the highest axial and lateral deformations.
Figure 4.3  Failure mechanisms of the specimens under 25 mm eccentric loading
Figure 4.4 Failure mechanisms of the specimens under 50 mm eccentric loading
For the specimens that were tested under 50 mm eccentric load, it can be seen from Figure 4.6 that all the specimens had the same behaviour before yield point. It can be recognized that Specimen PT-50 achieved the lowest maximum load compared to the other specimens due to the holes on the tube which reduced its confinement efficiency. Specimens ST-50 and ST-G-50 had mostly the same maximum load. Specimen REF-50 had the highest axial and lateral deformation.

Figure 4.5 Load-deflection diagram of specimens tested under 25 mm eccentricity
Figure 4.6 Load-deflection diagram of specimens tested under 50 mm eccentricity
4.4 Specimens tested under flexural

To evaluate the flexural behaviour of the GFRP tube-reinforced concrete specimens, four specimens were tested under four point loading, one specimen from each group.

4.4.1 Mechanism of the failure

For specimen REF-F, the concrete cover at the compression zone was crushed seriously during the test, however, with the increased midspan deflection, the specimen was able to carry higher load. After reaching its ultimate flexural load, Specimen REF-F failed by inclined shear cracks and flexural cracks. For Specimens PT-F, ST-F, and ST-G-F, the failure was due to the rupture of GFRP tube in the tension side accompanied with a load sound. For Specimens PT-F, and ST-G-F, the concrete cover almost spalled off because the bond strength between the concrete cover and the GFRP tube decreased significantly. In addition, the concrete cover crushed into small segments which can be easily removed. Nevertheless, for Specimen ST-G-F, the concrete cover was prevented from spalling by the polymer grid.

4.4.2 Load-deflection diagram

At the first stage of the diagram, all the specimens had the same axial load-midspan deflection behaviour. Afterwards, due to spalling of the concrete cover, load reduction can be observed in all specimens. After load reduction, load fluctuation was observed in all the specimens. Specimen REF-F had the highest axial load after spalling of the concrete cover. Figure 4.7 shows the midspan deflection behaviour of the specimens subjected to four point loading.
The increased performance of the concrete beam specimens may be to the failure in a manner of combination of inclined shear cracks and flexural cracks, or due to the confinement efficiency provided by the steel helix may be more effective than square or rectangular stirrups. After load reduction of Specimen ST-G-F, the specimen could be further loaded to sustain a higher load. The sudden reduction for Specimens ST-F and PT-F was too high from 311 kN to 266 kN and from 337 kN to 227 kN, respectively. Subsequently, the recovery of the load could not take place. After load reduction, a substantial amount of load could be carried by Specimens PT-F and ST-F with the increase of the mid-span deflection until failure. Specimen PT-F had the higher ultimate midspan deflection compared to Specimens ST-F and ST-G-f, which experienced a higher load carrying capacity compared to Specimen ST-F and PT-F. Table 4.3 summarises the test results of the specimens under flexural loading.
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield load (kN)</th>
<th>Maximum load (kN)</th>
<th>Mid span deflection at maximum load (mm)</th>
<th>Maximum Moment (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>REF-F</td>
<td>275</td>
<td>369</td>
<td>25.4</td>
<td>43</td>
</tr>
<tr>
<td>PT-F</td>
<td>238</td>
<td>311</td>
<td>8.8</td>
<td>36</td>
</tr>
<tr>
<td>ST-F</td>
<td>240</td>
<td>332</td>
<td>9.4</td>
<td>39</td>
</tr>
<tr>
<td>ST-G-F</td>
<td>270</td>
<td>348</td>
<td>23.3</td>
<td>40</td>
</tr>
</tbody>
</table>

### 4.5 Strain gauges readings

According to the readings from strain gauges, Specimens ST-0 and STG-0 failed at longitudinal compressive strains of 0.00645 and 0.00659, respectively. Even though higher ultimate loads could be observed for Specimens ST-0 and STG-0, the axial deformations at the ultimate loads were significantly less than that of Specimen REF-0. This phenomenon was attributed to the low hoop tensile properties of GFRP tubes. Therefore, Specimens ST-0 and STG-0 failed due to the hoop tensile rupture of GFRP tubes before the axial compressive strength of GFRP tubes can be fully utilized.

For Specimens ST-F and PT-F, the FRP tube ruptured immediately after the spalling of the concrete cover (the longitudinal compressive strains of FRP tube at rupture in the extreme compression fibre were 0.0025 and 0.0016), respectively, which resulted in sudden load reductions of specimens. After these load reductions from 337 kN to 227 kN for Specimen ST-F and from 311 kN to 266 kN for Specimen PT-F, the specimens could still carry a substantial amount of loads with increasing mid-span deflection until failure. For Specimen STG-F, the FRP tube ruptured at a longitudinal compressive strain of around 0.0045 in the extreme compression fibre. Therefore, Specimen STG-F could be further loaded to obtain higher load and undergo higher mid-span deflection after the spalling of concrete cover.
Figure 4.8 Failure mechanisms of the specimens tested under flexural loading
4.6 Effect of the eccentricity on the maximum load

Test results indicated that the maximum load for steel reinforced concrete columns SRCs and FRP tube reinforced concrete columns FTRCs decreased significantly with the increased eccentricity. Group REF which is SRCs, showed the highest drop in maximum load compared to Group ST, Group PT, and Group ST-G which are FTRCs. Figure 4.9 shows the effect of the eccentricity on the maximum load. For Group REF, the drop in the maximum load from axial load to 25 mm eccentric load is about 33%. For FTRCs, the highest drop in maximum load was for Group ST with 20% followed by Group ST-G with 16% drop. For Group PT the maximum load was the same for concentrically and 25 mm eccentrically loaded columns. This is because the specimen PT-0 had premature failure at the ends which reduces the maximum load. The results showed that the effect of the eccentricity on FTRCs is less than its effect on the SRCs.

![Figure 4.9 Maximum load - eccentricity diagrams](image)

4.7 Axial load - bending moment interaction diagram

For concentrically loaded columns, it is better to evaluate the axial load – bending moment interaction behaviour. The axial load – bending moment \((P-M)\) diagram represents a plot of axial load against bending moment, and the inverse of the eccentricity represents the slope of
the loading line. As a result, the section capacity of the specimens can be evaluated by using P-M diagram, so that specimens can resist a combination of axial load and bending moment. For concentrically loaded column, the experimental P-M diagram can be drawn directly by using the maximum load in different eccentricity. For eccentrically loaded columns, the bending moment can be evaluated by Equation 4.1.

\[ M = P(e + \delta) \]  \hspace{1cm} (4.1)

Where \( P \) is the maximum load, \( e \) is the test eccentricity and \( \delta \) is the lateral deformation of the specimen at maximum load. Figure 4.10 shows the schematic diagram for columns subjected to eccentric loading.

![Figure 4.10 Eccentrically loaded column](image)

For specimens subjected to flexural, the calculations of the moment capacity can be done by using Equation 4.2

\[ M = \frac{P_x}{2} \]  \hspace{1cm} (4.2)
Where $P$ is the maximum load for the specimen under flexural loading, $x$ is the shear span and it can represent the distance between the outer most loading point on the bottom loading plate and the outer most loading point on the top loading plate on the same side. The test results and calculated moment for $P$-$M$ diagram are summarised in Table 4.4. The experimental interaction diagrams are shown in Fig. 4.11. The interaction diagrams indicate that FTRC specimens (Groups ST, ST-G, and PT) outperformed the SRC specimens. A balance point can be observed for the interaction diagrams of FTRC specimens. Below the balance point, the axial load increased with the increase of bending moment, which corresponds to the rupture of GFRP tube in the tension side. Above the balance point, the axial load increased with the decrease of bending moment, which corresponds to the rupture of GFRP tube in the compression side. The interaction diagram of steel RC specimens Group REF was not as expected since the axial load increased with a continuous decrease of bending moment. This phenomenon was due to the fact that Specimen REF-F failed by a combination of flexural and shear failure, Figure 4.8 (a). The shear cracks resulted from the arch action, which can result in an increase of the bending moment. Also, the size effect cannot be ignored because the size of beam specimen is rather small which may not fully reflect the actual flexural behaviour of the specimens.

![Figure 4.11 Experimental P-M diagrams](image)
Table 4.4 Summary of the test results for P-M diagram

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Eccentricity (mm)</th>
<th>Lateral Deflection (mm)</th>
<th>Maximum load (kN)</th>
<th>Bending Moment (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>REF-0</td>
<td>0</td>
<td>0</td>
<td>1486</td>
<td>0</td>
</tr>
<tr>
<td>REF-25</td>
<td>25</td>
<td>1.4</td>
<td>986</td>
<td>26</td>
</tr>
<tr>
<td>REF-50</td>
<td>50</td>
<td>2.3</td>
<td>6966</td>
<td>36</td>
</tr>
<tr>
<td>REF-F</td>
<td>Flexural</td>
<td>-</td>
<td>369</td>
<td>43</td>
</tr>
<tr>
<td>ST-0</td>
<td>0</td>
<td>0</td>
<td>1850</td>
<td>0</td>
</tr>
<tr>
<td>ST-25</td>
<td>25</td>
<td>5.1</td>
<td>1477</td>
<td>44</td>
</tr>
<tr>
<td>ST-50</td>
<td>50</td>
<td>5.7</td>
<td>915</td>
<td>51</td>
</tr>
<tr>
<td>ST-F</td>
<td>Flexural</td>
<td>-</td>
<td>337</td>
<td>39</td>
</tr>
<tr>
<td>PT-0</td>
<td>0</td>
<td>0</td>
<td>1414</td>
<td>0</td>
</tr>
<tr>
<td>PT-25</td>
<td>25</td>
<td>7.2</td>
<td>1318</td>
<td>42</td>
</tr>
<tr>
<td>PT-50</td>
<td>50</td>
<td>7.5</td>
<td>1038</td>
<td>59</td>
</tr>
<tr>
<td>PT-F</td>
<td>Flexural</td>
<td>-</td>
<td>311</td>
<td>36</td>
</tr>
<tr>
<td>ST-G-0</td>
<td>0</td>
<td>0</td>
<td>1849</td>
<td>0</td>
</tr>
<tr>
<td>ST-G-25</td>
<td>25</td>
<td>4.9</td>
<td>1558</td>
<td>46</td>
</tr>
<tr>
<td>ST-G-50</td>
<td>50</td>
<td>6.7</td>
<td>1045</td>
<td>59</td>
</tr>
<tr>
<td>ST-G-F</td>
<td>Flexural</td>
<td>-</td>
<td>309</td>
<td>36</td>
</tr>
</tbody>
</table>

4.8 Ductility

Ductility can be defined as the ability of significant deformation achievement before failure. The ductility is a wanted character in any concrete structure, because it helps to indicate people warning before failure. Furthermore, ductility protects structural members from failure due to unpredicted loads such as earthquakes. There are many methods to calculate ductility such as Pessiki and Pieroni (1997). This method calculates the ductility by Equation 4.3 which is the ratio of the displacement at \(0.85P_{\text{max}}\) (post peak load) to an initial displacement which represents approximate elastic behaviour limit displacement, as can be seen in Figure 4.12.
\[ \mu = \frac{\Delta_{0.85}}{\Delta_1} \]  

Figure 4.12 A Method of calculating ductility (Pessiki and Pieroni 1997)

Where \( \Delta_{0.85} \) displacement at 0.85\( P_{\text{max}} \) (post peak load), and \( \Delta_1 \) is an initial displacement corresponding to the intersection of the best fit line to the linear portion of the diagram with the maximum load.

Ductility calculations shows that the FRP tube reinforced concrete columns (FTRC) have higher ductility compared to the steel reinforced concrete columns (SRC); however, the load-deformation diagram shows that the SRC has a higher deformation compared to (FTRC), as can be seen in Table 4.5. Therefore, another method is used to calculate the ductility by calculating the toughness value. This method was firstly suggested by Smart and Jensen (1997). Toughness value can be calculated by integrating the total area under the load-deformation curve, measuring the energy absorbed by the columns during failure. As can be seen in Table 4.6, this method gives results different from pervious method. The ductility for the SRC columns is higher than FTRC columns, and it increases with the increase of the eccentricity, as can be seen in Figure 4.13. For the same loading conditions, the FTRC
columns mostly have the same ductility with small differences. In general, the perforated FRP tube reinforced concrete columns have the lowest ductility compared to the other FTRC columns, because of the perforations in the tube which decrease the deformation capacity for the columns.

Table 4.5 ductility calculations according to Pessiki and Pieroni (1997) method

(a) Concentrically loaded columns

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Load eccentricity (mm)</th>
<th>$0.85\Delta_{\text{ultimate}}$</th>
<th>$\Delta_{\text{yield}}$</th>
<th>Ductility = $\frac{0.85\Delta_{\text{Ultimate}}}{\Delta_{\text{Yield}}}$</th>
<th>Normalized ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>REF-0</td>
<td>0</td>
<td>3.61</td>
<td>2.20</td>
<td>1.64</td>
<td>1</td>
</tr>
<tr>
<td>PT-0</td>
<td></td>
<td>5.50</td>
<td>3.14</td>
<td>5.5</td>
<td>3.27</td>
</tr>
<tr>
<td>ST-0</td>
<td></td>
<td>6.98</td>
<td>2.01</td>
<td>3.47</td>
<td>2</td>
</tr>
<tr>
<td>ST-G-0</td>
<td></td>
<td>7.73</td>
<td>2.68</td>
<td>2.88</td>
<td>1.7</td>
</tr>
</tbody>
</table>

(b) Columns tested under eccentric load (25 mm)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Load eccentricity (mm)</th>
<th>$0.85\Delta_{\text{ultimate}}$</th>
<th>$\Delta_{\text{yield}}$</th>
<th>Ductility = $\frac{0.85\Delta_{\text{Ultimate}}}{\Delta_{\text{Yield}}}$</th>
<th>Normalized ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>REF-25</td>
<td>25</td>
<td>2.85</td>
<td>2.87</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>PT-25</td>
<td></td>
<td>7.55</td>
<td>3.20</td>
<td>2.36</td>
<td>2.36</td>
</tr>
<tr>
<td>ST-25</td>
<td></td>
<td>7.16</td>
<td>3.21</td>
<td>2.23</td>
<td>2.23</td>
</tr>
<tr>
<td>ST-G-25</td>
<td></td>
<td>7.07</td>
<td>3.05</td>
<td>2.31</td>
<td>2.31</td>
</tr>
</tbody>
</table>

(c) Columns tested under eccentric load (50 mm)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Load eccentricity (mm)</th>
<th>$0.85\Delta_{\text{ultimate}}$</th>
<th>$\Delta_{\text{yield}}$</th>
<th>Ductility = $\frac{0.85\Delta_{\text{Ultimate}}}{\Delta_{\text{Yield}}}$</th>
<th>Normalized ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>REF-50</td>
<td>50</td>
<td>2.96</td>
<td>1.9</td>
<td>1.56</td>
<td>1</td>
</tr>
<tr>
<td>PT-50</td>
<td></td>
<td>9.5</td>
<td>3.20</td>
<td>3</td>
<td>1.92</td>
</tr>
<tr>
<td>ST-50</td>
<td></td>
<td>10.20</td>
<td>2.56</td>
<td>3.97</td>
<td>2.54</td>
</tr>
<tr>
<td>ST-G-50</td>
<td></td>
<td>9.24</td>
<td>3.20</td>
<td>2.88</td>
<td>1.84</td>
</tr>
</tbody>
</table>
Table 4.6 Smart and Jensen (1997) method for toughness calculations

(a) Concentrically loaded columns

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Load eccentricity (mm)</th>
<th>Ductility= Area under the curve (kN.mm)</th>
<th>Normalized ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>REF-0</td>
<td>0</td>
<td>35166</td>
<td>1</td>
</tr>
<tr>
<td>PT-0</td>
<td>0</td>
<td>6998</td>
<td>0.20</td>
</tr>
<tr>
<td>ST-0</td>
<td>0</td>
<td>8801</td>
<td>0.25</td>
</tr>
<tr>
<td>ST-G-0</td>
<td>0</td>
<td>10157</td>
<td>0.29</td>
</tr>
</tbody>
</table>

(b) Columns tested under eccentric load (25 mm)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Load eccentricity (mm)</th>
<th>Ductility= Area under the curve (kN.mm)</th>
<th>Normalized ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>REF-25</td>
<td>25</td>
<td>11569</td>
<td>1</td>
</tr>
<tr>
<td>PT-25</td>
<td>25</td>
<td>7691</td>
<td>0.67</td>
</tr>
<tr>
<td>ST-25</td>
<td>25</td>
<td>8363</td>
<td>0.72</td>
</tr>
<tr>
<td>ST-G-25</td>
<td>25</td>
<td>8489</td>
<td>0.73</td>
</tr>
</tbody>
</table>

(c) Columns tested under eccentric load (50 mm)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Load eccentricity (mm)</th>
<th>Ductility= Area under the curve (kN.mm)</th>
<th>Normalized ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>REF-50</td>
<td>50</td>
<td>9001</td>
<td>1</td>
</tr>
<tr>
<td>PT-50</td>
<td>50</td>
<td>7050</td>
<td>0.78</td>
</tr>
<tr>
<td>ST-50</td>
<td>50</td>
<td>9244</td>
<td>1.02</td>
</tr>
<tr>
<td>ST-G-50</td>
<td>50</td>
<td>6933</td>
<td>0.77</td>
</tr>
</tbody>
</table>
Figure 4.13 Normalized ductility for specimens by using Strain-energy method

4.9 Summary

FTRC column specimens Group ST, ST-G, and PT can obtain higher load carrying capacity compared to Group REF column specimens under both concentric and eccentric loadings. The maximum load of FTRC column specimens is significantly reduced with the increase of eccentricity. Group ST-G column specimens achieve the highest maximum load, followed by Groups ST, PT, and REF column specimens. In addition, the ductility of FTRC column specimens is less than the ductility of Group REF column specimens under both concentric and eccentric loading conditions. Furthermore, among the four beam specimens REF-F, ST-F, ST-G-F, and PT-F, Specimen REF-F had the highest maximum load, followed by Specimens ST-G-F, ST, and PT. The highest midspan deflection was obtained by Specimen REF-F, followed by Specimens PT, ST-G, and ST. Finally, interaction ($P$-$M$) diagrams are constructed to show the enhanced performance of FTRC specimens than the reference SRC specimens. The theoretical analysis of the data is presented in the following chapter.
Chapter 5 **Theoretical Analysis**

5.1 **Introduction**

Most of the compression members in concrete structures are subjected to a combination of axial load and bending moment due to eccentric load. Different reasons cause the eccentric loading such as, misalignment of load, or horizontal loads like earthquake and wind. In the design and analysis of concrete columns, the estimation of the axial load-bending moment (P-M) interaction behaviour is very critical. According to Warner *et al.* (2007), the P-M diagram for unconfined concrete can be drawn by using four points.

Point A: pure axial load point or squash load point, in this point the column is only subjected to pure axial load without moment.

Point B: this point has an axial load and bending moment when the neutral axis depth is equal to the effective depth of the column which means the neutral axis is at the tensile reinforcement.

Point C: Balanced failure point when the concrete start to crush (compressive strain equal to 0.003) at the same time of the yield of the steel reinforcement.

Point D: In this point there is no axial load, just pure bending moment.

This chapter presents an evaluation of the theoretical $P-M$ datagram for steel reinforced concrete specimens and concrete filled FRP tube specimens CFTTs. Finally, a comparison between the experimental and theoretical $P-M$ interaction diagrams is presented.

5.2 **Assumptions**

General assumptions were proposed by AS3600 (2009) for establishing an interaction diagram.

1. Plane sections normal to the axis remain plane after bending.
2. The tensile strength of concrete is neglected.

3. The distribution of stresses in the concrete and steel is determined using stress-strain relationships.

4. The strain in the compressive steel reinforcement does not exceed 0.003.

5. Where the neutral axis lies outside of the cross-section, consideration shall be given to the effect on strength of spalling of the cover concrete.

5.3 P-M Interaction diagram for steel reinforced concrete specimens

For the calculation of the theoretical P-M Interaction diagram of steel reinforced concrete specimens, equivalent rectangular stress block method is used as can be seen in Figure A.1 in Appendix A. According to Wight and Macgregor (2011), five points have been used to draw the theoretical interaction diagram. The first point is the squash point which represents axial load without bending moment. The second point, the strain at the compressive steel reinforcement is equal to zero. The third point, the strain at compressive steel reinforcement is equal to the yield strain of the steel reinforcement. The fourth point, the strain at the compressive steel reinforcement is equal to 0.005. Finally, the last point is the pure bending moment point.

Figure 5.1 shows the theoretical P-M interaction diagram for steel reinforced specimens by using equivalent stress block method. The steps of calculation theoretical P-M diagram for steel reinforced specimens are described in appendix A at the end of the thesis.
Figure 5.1 Axial load-bending moment P-M diagram for steel reinforced specimens

5.4 Axial load-bending moment interaction diagram for CFFTs

For confined concrete, the depth of the neutral axis is hard to be determined at balanced failure. A theoretical interaction diagram has been drawn for CFFTs based on four points (a) Point A, squash load point without moment; (b) Point B is taken at load eccentricity of $e = 25$ mm; (c) Point C is taken at load eccentricity of $e = 50$ mm; (d) Point D, pure bending point. For CFFTs, the contribution of concrete cover was neglected since most of the concrete cover spalled off at the time of failure.

5.4.1 Confinement model

Lam and Teng (2003a) design oriented stress-strain model has been used widely in practical application for FRP confined concrete columns. ACI 440.2R (2008) adopted Lam and Teng (2003a) for FRP confined concrete. The stress-strain model is based on following assumptions.

1. The first portion of the stress-strain curve is parabolic and the second portion is straight.
2. At $\varepsilon_t = 0$, the parabola has the same slope as elastic modulus of unconfined concrete ($E_c$).
3. The presence of FRP jacket affects the nonlinear part of the first portion by some degree.
4. The parabolic first portion meets with the linear second portion smoothly.

5. When the confined concrete reaches its compressive strength and the ultimate axial strain, the linear second portion ends.

![Figure 5.2 FRP confined concrete Stress-strain model (Lam and Teng 2003a)](image)

The stress-strain model for FRP confined concrete is proposed based on these assumptions and is given by the following expressions.

\[
\sigma_c = E_c \varepsilon_c - \frac{(E_c - E_2)^2}{4f'_{co}} \varepsilon_c^2 \quad \text{For } 0 \leq \varepsilon_c \leq \varepsilon_t \quad (5.1)
\]

\[
\sigma_c = f'_{co} + E_2 E_c \quad \text{For } \varepsilon_t \leq \varepsilon_c \leq \varepsilon_{cu} \quad (5.2)
\]

Where \( \sigma_c \) and \( \varepsilon_c \) are the compressive stress and strain, respectively. \( f'_{co} \) is the compressive strength of unconfined concrete, \( E_c \) is the elastic modulus of unconfined concrete and \( E_2 \) is the slope of the linear second portion given by Equation 5.3. The transition strain between the parabolic and linear portion \( \varepsilon_t \) is given by Equation 5.4.
\[ E_2 = \frac{f'_{cc} - f'_{co}}{\varepsilon_{cu}} \] 

(5.3)

Where \( f_{cc} \) is the compressive strength of confined concrete, and \( \varepsilon_{cu} \) is the ultimate strain of confined concrete.

\[ \varepsilon_t = \frac{2f'_{co}}{(E_c - E_2)} \] 

(5.4)

\[ \frac{f_{cc}}{f'_{co}} = 1 + 3.3 \frac{f_t}{f'_{co}} \] 

(5.5)

Where \( f_t \) is the effective confining pressure of FRP, which can be calculated by Eq. 5.6.

\[ f_t = \frac{2f_{frp} t_f}{d} \] 

(5.6)

Where \( f_{frp} \) is the rapture stress of FRP tube, \( t_f \) is the thickness of FRP tube, \( D \) is the diameter of the column.

As mentioned above, four points have been used to draw the theoretical P-M interaction diagram, squash load point, Point B taken load eccentricity of 25 mm, Point C taken at load eccentricity of 50 mm, and pure bending moment point. For the calculation of the squash load, the following Equation 5.7 is used.

\[ P_u = 0.85A_c f_{cc} + E_{frp} A_{frp} \varepsilon_{cu} \] 

(5.7)

Where \( A_c \) is the area of core concrete, \( E_{frp} \) Modulus of Elasticity of FRP tube in longitudinal direction, and \( A_{frp} \) is the area of FRP tube.
Cole and Fam (2006), adopted a layer by layer method for the integration of the stresses over the concrete and GFRP tube cross sectional areas. Figure 5.3 shows a layer by layer method. The cross section area of the concrete filled FRP tube CFTTs was divided into small stripes with thickness of $i$ and $n$ number. Each strip consists of two different materials, GFRP tube and concrete. The mid thickness of each strip represents its centroid and the depth from the top level to the centroid of the strips represents $h_i$. The bond between the GFRP tube and concrete is assumed as a perfect bond and the distribution of the strains is linear along the section depth. A linear elastic stress-strain relationship is adopted for the GFRP tube in both longitudinal and hoop direction. A confinement model of Lam and Teng (2003a) is adopted for the calculations of the compressive stress of CFTTs. Furthermore, the actual longitudinal compressive strain of FRP tube in the extreme compressive fibre is used as ultimate axial compressive strain $\varepsilon_{cu}$ of confined concrete. The centroid stresses throughout the thickness of the strips is depending on the assumption of constant stresses. The strain at each strip $\varepsilon_i$ at depth $h_i$ can be calculated by using Equation 5.8. Figure 5.4 shows area calculations for each strip. Table 5.1 shows the strain gauges reading and the elastic modulus that have been used in the calculations.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>ST-0</th>
<th>ST-25</th>
<th>ST-50</th>
<th>ST-G-0</th>
<th>ST-G-25</th>
<th>ST-G-50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial compressive strain at failure</td>
<td>0.00645</td>
<td>0.0105</td>
<td>0.0104</td>
<td>0.00659</td>
<td>0.0121</td>
<td>0.0116</td>
</tr>
<tr>
<td>Hoop tensile strain at failure</td>
<td>0.0047</td>
<td>0.0043</td>
<td>0.0039</td>
<td>0.0057</td>
<td>-</td>
<td>0.004</td>
</tr>
<tr>
<td>Elastic modulus (GPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>28.7</td>
</tr>
</tbody>
</table>
\[ \varepsilon_t = \left( \frac{d_n - (n - 0.5)}{d_n} \right) \varepsilon_{cu} > 0 \]  \hspace{1cm} (5.8)

Where \( d_n \) is the depth of the neutral axis, \( \varepsilon_{cu} \) is the actual longitudinal compressive strain of FRP tube in the extreme compressive fibre. Figure 5.5 shows a flowchart that explains the steps of obtaining the theoretical (\( P-M \)) diagram for CFFTs.

\[ A_c = 2 \sqrt{r_c^2 - (r_t - (n - 0.5))^2} \]
\[ A_{frp} = 2 \sqrt{r_t^2 - (r_t - (n - 0.5))^2} \]

Figure 5.3 Layer by layer method

Figure 5.4 Area calculations for each strip
Figure 5.5 Flow chart for the calculations of theoretical P-M interaction diagram for CFFT's

1. Assume \( d_n \)
2. Calculate area of concrete \( A_c \) AND Area of FRP tube \( A_{frp} \)
3. Calculate \( \varepsilon_{ci}, f_{frp}, P_{frp} \)
4. Calculate \( \varepsilon_t \)
5. If \( \varepsilon_{ci} < \varepsilon_t \)
   - Yes: Use Equation 5.1
   - No: Use Equation 5.2
   - \( P_c = A_c\sigma_c \)
6. Calculate \( P_{total}, M_c, M_{frp}, M_{total} \)
7. \( \varepsilon_l = \frac{M_{total}}{P_{total}} \)
8. If \( \varepsilon_l = 25 \) or \( 50 \)
   - Yes: The calculated P and M are under given \( \varepsilon_l \)

Tube diameter \((d_{np})\), Tube thickness \((t_{np})\), FRP tube properties, \( n \), and \( \varepsilon_{cu} \) from Strain gauges
The flow chart process can be explained briefly as follows:

1. The inputs are the diameter of GFRP tube $d_{frp}$, thickness of the GFRP tube $t_{frp}$, and the ultimate axial compressive strain $\varepsilon_{cu}$ of confined concrete which is the actual longitudinal compressive strain of FRP tube in the extreme compressive fibre.

2. Assume the depth of neutral axis $d_n$.

3. Calculate the area of concrete $A_c$ and the Area of the GFRP tube $A_{frp}$ for each strip.

4. Calculate the strain in each strip, the rapture stress of FRP tube $f_{frp}$, the transition strain $\varepsilon_t$ and the compressive force of GFRP tube $P_{frp}$.

5. If $\varepsilon_{ci}$ is less than $\varepsilon_t$, use Equation 5.1 for the calculation of the compressive stress of confined concrete $\sigma_c$, otherwise, use Equation 5.2.

6. Calculate the compressive force of concrete $P_c$ by multiplying the compressive stress of concrete by the area of concrete.

7. Calculate the total force $P_T$, moment of the concrete $M_c$, moment of GFRP tube $M_{frp}$ and the total moment $M_T$.

8. Calculate the eccentricity $e_i$ by dividing total moment by the total force.

9. If the eccentricity is equal to 25 mm or 50 mm, the calculated P and M are under given $e_i$.

### 5.5 Validation with the experimental results

For verification of the experimental results, theoretical axial load-bending moment P-M interaction diagram is constructed. Equivalent rectangular stress block method is used for the steel reinforced specimens SRCs and a layer by layer method is used for CFFT specimens. Lam and Teng (2003a) model is proposed for the calculation of the stresses in the FRP confined concrete. In general, the experimental and theoretical P-M interaction diagrams exhibit the same patterns except for Group REF. Figure 5.6 shows the theoretical P-M
interaction diagrams for all Groups. Table 5.2 shows a summary of results for theoretical P-M interaction diagram.

![Theoretical P-M interaction diagrams for all groups](image)

Figure 5.6 Theoretical P-M interaction diagrams for all groups

A separate comparison between theoretical and experimental P-M interaction diagrams for each group is presented.
Table 5.2 Summary of results for theoretical P-M diagram

<table>
<thead>
<tr>
<th>Group</th>
<th>Point</th>
<th>Maximum load (kN)</th>
<th>Bending moment (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Point A: pure axial load</td>
<td>1665</td>
<td>0</td>
</tr>
<tr>
<td>REF</td>
<td>Point B: $\varepsilon_{st} = 0$</td>
<td>1050</td>
<td>37.48</td>
</tr>
<tr>
<td></td>
<td>Point C: $\varepsilon_{st} = \varepsilon_y$</td>
<td>361</td>
<td>41.21</td>
</tr>
<tr>
<td></td>
<td>Point D: $\varepsilon_{st} = 0.005$</td>
<td>91</td>
<td>30.76</td>
</tr>
<tr>
<td></td>
<td>Point E: Pure bending moment</td>
<td>0</td>
<td>24.92</td>
</tr>
<tr>
<td>ST</td>
<td>Point A: pure axial load</td>
<td>1794</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Point B: Zero tension</td>
<td>1581</td>
<td>39.52</td>
</tr>
<tr>
<td></td>
<td>Point C: Balanced failure</td>
<td>1058</td>
<td>52.90</td>
</tr>
<tr>
<td></td>
<td>Point D: Pure bending</td>
<td>0</td>
<td>28.61</td>
</tr>
<tr>
<td>ST-G</td>
<td>Point A: pure axial load</td>
<td>1816</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Point B: Zero tension</td>
<td>1440</td>
<td>36</td>
</tr>
<tr>
<td></td>
<td>Point C: Balanced failure</td>
<td>985</td>
<td>49.25</td>
</tr>
<tr>
<td></td>
<td>Point D: Pure bending</td>
<td>0</td>
<td>28.61</td>
</tr>
</tbody>
</table>

For Group REF, theoretical P-M interaction diagram overestimates the axial load carrying capacity about 12% compared to the experimental axial load. On the other hand, theoretical P-M interaction diagram underestimates bending moment about 76% compared to experimental bending moment. The big difference may because of failure mechanism of the REF beam which was due to a combination of flexural and shear failure. Figure 5.7 shows a comparison between theoretical and experimental P-M diagrams for REF Group.
For Group ST, the theoretical P-M interaction diagram is in a good fit with the experimental P-M interaction diagram in concentric and eccentric loading with a small difference in concentric load about 12.3% under estimation. The theoretical prediction is underestimates bending moment about 28.5% compared to experimental bending moment. Figure 5.8 shows a comparison between the theoretical and experimental P-M interaction diagrams for Group ST.
The theoretical prediction for Group ST-G is much better than Group ST, it is in a good fit with the experimental P-M interaction diagram in all loading cases, concentric, eccentric and flexural loading with a small difference in concentric load about 11% under estimation. Figure 5.9 shows a comparison between the theoretical and experimental P-M interaction diagrams for Group ST-G.

In general, in comparison with the experimental P-M interaction diagrams, the theoretical P-M interaction diagrams for CFFTs specimens is much appropriate than the theoretical P-M interaction diagrams for SRCs.

![Figure 5.9 Comparison between the theoretical and experimental P-M interaction diagrams for Group ST-G](image)

**5.6 Summary**

A theoretical analysis on the P-M behaviour of steel reinforced concrete specimens SRCs and concrete filled FRP specimens CFFTs was conducted in this chapter. An equivalent rectangular stress block method is used for SRCs and layer by layer method is used for CFFTs. A comparison between the theoretical P-M behaviour and experimental P-M behaviour which presented in chapter 4, is carried out for Group REF, Group ST, and Group ST-G separately. The theoretical P-M prediction for CFFTs provides a reasonable
comparison with the experimental P-M behaviour, while for SRCs, it over estimates the concentric and eccentric loading and under estimate the bending moment. Finally, the discussion and conclusion for this study are presented in the following chapter.
Chapter 6 Conclusions

6.1 Introduction

A study about FRP tube reinforced concrete columns FTRCs has been presented in this study. Sixteen specimens, which contained four steel reinforced specimens SRCs and twelve FTRCs specimens were cast and tested in this study. The load carrying capacity, failure mode and ductility were examined. In addition, axial load-bending moment (P-M) interaction behaviour was evaluated. Analytical evaluation of P-M behaviour by using (Lam and Teng 2003a) was adopted.

6.2 Conclusions

Based on the experimental and theoretical examination the following conclusion can be drawn:

1. FTRC specimens which include Groups ST, ST-G and PT exhibited higher load carrying capacity compared to SRC specimens under concentric and eccentric loading. The load carrying capacity is decreased significantly as the eccentricity increased. The column specimens for Group ST-G achieved the highest load carrying compared to the specimens in the other groups followed by Group ST, Group PT and Group REF.

2. The effect of the eccentricity was examined in this study. The maximum load for all specimens decreases with the increase of the eccentricity. The highest drop in maximum load is for Group REF with drop of a 33%, followed by Group ST and Group ST-G.

3. Two methods have been used for the calculations of the ductility. The first one is Pessiki and Pieroni (1997) method by dividing the deformation corresponding to 0.85 $p_{max}$ over the deformation corresponding to the yield load. Results show that the ductility for FTRCs specimens is higher than the ductility for SRCs specimens under concentric and eccentric loading. Another method is used to calculate the ductility by using strain-energy method.
Results show that the ductility for SRCs specimens under concentric and eccentric loading is higher than the ductility for FTRCs specimens. For both methods, the ductility increased as the load eccentricity increased.

4. For beams, Specimen REF-F had the highest load carrying capacity compared to the other specimens followed by Specimens ST-G-F, ST-F and PT-F. In addition, Specimen REF-F has the highest mid-span deflection compared to other specimens followed by Specimens PT-F, ST-G-F and ST-F.

5. P-M interaction diagram has been constructed to evaluate P-M behaviour of FTRC specimens compared to SRC specimens. Results show that FTRCs specimens exhibit higher axial load-bending moment capacity than SRCs specimens. For pure bending moment, the SRCs specimens exhibit higher bending moment.

6.3 Future studies

The following suggestions are the areas that can be covered in the future studies:

1. This study is based on 16 specimens; therefore, more experimental program can be conducted to validate the conclusions of this study.

2. The same study can be conducted with different cross-section (square, rectangle).

3. The behaviour of FTRC columns under severe conditions (freeze, and high temperature) could be conducted.

4. The influence of fibre orientation on the performance of FTRC columns under different loading conditions could be further investigated.
References


Campione, G. & Miraglia, N. 2003, 'Strength and strain capacities of concrete compression members reinforced with FRP', *Cement and Concrete Composites*, vol. 25, no. 1, pp. 31-41.


Ozbakkaloglu, T. & Oehlers, D. J. 2008, 'Concrete-filled square and rectangular FRP tubes under axial compression', *Journal of Composites for Construction*, vol. 12, no. 4, pp. 469-477.


Appendix A: Calculation of the theoretical P-M interaction diagram for steel reinforced specimens by using stress block method

A.1 Point A: squash load

According to Warner et al. (2007) the squash load point can be calculated by using Equation A.1, by adding a maximum force of the steel and concrete.

\[ P_u = \alpha_1 f'_c (A_g - A_s) + A_s f_{sy} \] (A.1)

Where \( f'_c \) is the compressive strength of concrete at 28 days. \( A_g \) is gross area of the concrete, \( A_s \) is the total area of the longitudinal steel reinforcement, \( \alpha_1 \) is a reduction factor according to AS3600 (2009) and it is calculated by Equation A.2.

\[ \alpha_1 = 1 - 0.003 f'_c \] Where \( 0.72 \leq \alpha_1 \leq 0.85 \) (A.2)

A.2 Points B, C and D

According to Wight and Macgregor (2011), the compression part of the column is piece of the circular column. Compression zone area and moment can be calculated by using Equations A.3 and A.4.

\[ A = h^2 \left( \frac{\theta \text{ rad} - \sin \theta \cos \theta}{4} \right) \] (A.3)

\[ A_y = h^3 \left( \frac{\sin^2 \theta}{12} \right) \] (A.4)

Where \( h \) is the diameter of the specimen, \( \theta \) is angle and it is in radians. Angle \( \theta \) can be calculated by using Equation A.5 and A.6.
\[ \theta = \cos^{-1} \theta \left( \frac{h}{\frac{h}{2}} - \frac{a}{h} \right) \quad \text{For} \ h \leq \frac{a}{2} \quad (A.5) \]

\[ \theta = 180 - \cos^{-1} \theta \left( \frac{h}{\frac{h}{2}} - \frac{a}{h} \right) \quad \text{For} \ h > \frac{a}{2} \quad (A.6) \]

Where \( a \) is compression zone depth, \( y \) is the distance between column centroid and compression zone centroid.

As a result, the compressive axial load and moment can be calculated by using Equations A.7 and A.8.

\[ C_c = \alpha_1 f'_c A \quad (A.7) \]

\[ M_c = \alpha_1 f'_c Ay \quad (A.8) \]

Therefore, the total axial load and moment can be calculated by using (Warner et al. (2007)) Equation A.9 and A.10

\[ P = C_c + C_s - T \quad (A.9) \]

\[ M = M_c + C_s \left( \frac{d'}{2} - d_c \right) - T(\frac{d'}{2} - \frac{d}{2}) \quad (A.10) \]

Where \( d_o \) is the distance between the compressive reinforcement centre and the outmost compression fibre, \( d' \) is the distance between the centre of the tensile reinforcement and the outmost compression fibre. Similar triangles method can be used to calculate the strain in steel as can be seen in Figure A.1(b).
\[ \varepsilon_{sc} = \varepsilon_{cu} \left( \frac{d_n - d_\circ}{d_n} \right) \]  
(A.11)

Where \( \varepsilon_{cu} \) is the ultimate strain which equal to 0.003 for unconfined concrete and equal to the rupture of the steel straps for the steel reinforced concrete columns, \( d_n \) is neutral axis depth. In addition, the steel reinforcement stress can be calculated by Equation A.12.

\[ \sigma_{sc} = E_s \varepsilon_{sy} \quad \text{For } \varepsilon_{sc} < \varepsilon_{sy} \]  
(A.12)

\[ \sigma_{sc} = f_{sy} \quad \text{For } \varepsilon_{sc} \geq \varepsilon_{sy} \]  

Figure A.1 Equivalent rectangular stress block method

Where \( \varepsilon_{cu} \) is the ultimate strain which equal to 0.003 for unconfined concrete and equal to the rupture of the steel straps for the steel reinforced concrete columns, \( d_n \) is neutral axis depth. In addition, the steel reinforcement stress can be calculated by Equation A.12.
Where $\varepsilon_{xy}$ is the yield strain of the steel, $E_s$ is the elastic modulus of the steel and, $f_{sy}$ is the yield stress of the steel. As a result, the force in the compression reinforcement can be calculated by using Equation A.13.

$$C_s = \sigma_{sc}A_{sc} \quad \text{(A.13)}$$

$A_{sc}$ is the steel reinforcement area at the compression zone. The stresses and the strain at the tension reinforcement can be calculated by using Equations A.14 and A.15.

$$\varepsilon_{st} = 0.003\left(\frac{d'-d_n}{d_n}\right) \quad \text{(A.14)}$$

$$\sigma_{st} = E_s\varepsilon_{st} \quad \text{For } \varepsilon_{st} < \varepsilon_{sy} \quad \text{(A.15)}$$

$$\sigma_{st} = f_{sy} \quad \text{For } \varepsilon_{st} \geq \varepsilon_{sy}$$

Therefore, the force of the tensile reinforcement can be given by equation A.16.

$$T = \sigma_{st}A_{st} \quad \text{(A.16)}$$

Where $A_{st}$ is the area of the steel reinforcement at the tension zone.