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Behavior of carbon FRP strengthened T-connections under cyclic loading

Reza Pakfetrat
University of Wollongong

UNIVERSITY OF WOLLONGONG

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Reza PAKFETRAT

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Statement of Original Authorship

I, Reza Pakfetrat, hereby declare that the work contained in this thesis has not been previously submitted to meet requirements for an award at this or any other higher education institution. To the best of my knowledge and belief, the thesis contains no material previously published or written by another person except where due reference is made.
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Abstract

This study is an investigation into the efficacy of using carbon fiber reinforced polymer sheets as a strengthening technique for reinforced concrete beam-column connections after circularizing the columns in the vicinity of the beams. Moreover, the consequences of applying multiple carbon fiber reinforced polymer layers rather than a single layer as external reinforcement for beam-column connections are explored. In this study, three identical beam-column connections were cast. One specimen was used as the control specimen while the other two were externally reinforced in the joint area with CFRP sheets. Prior to the application of CFRP sheets to the joints, the cross sections of the columns in the vicinity of the joints for two of the specimens were circularized by attaching concrete segments in order to eliminate sharp corners in the cross-section in order to put to the test the theory that the presence of sharp corners increases the concentration of stress in the CFRP reinforcement which in turn triggers early debonding of CFRP sheets. One of the strengthened specimens was reinforced with six times the number of CFRP layers used for the other strengthened specimen to investigate the effects of composite density on the final strength. The specimens were then placed under displacement control cyclic loading.

The results of the tests confirmed the hypothesis that using CFRP composites as an external reinforcement technique for beam-column connections, significantly improves the performance of the joint under cyclic loading conditions. The ultimate strengths of the reinforced specimens were much higher than that of the control specimen and the strengthened specimens exhibited higher ductility than the control specimen. The results also suggested that the application of multiple CFRP layers enhanced joint ductility, ultimate strength and the overall performance of the specimens.
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Chapter 1: Introduction

1.1. Preamble

Beam-column connections are considered to play a significant role in the overall integrity of structures, yet at the same time, they have exhibited to be one of the weakest elements in structures under seismic loading conditions. Since the mid-1960s, the performance of beam-column joints has been studied as a significant issue in the seismic resistance of Reinforced Concrete Moment Resisting Frames (Hanson et al. 1967). Various strengthening techniques have been proposed for T-connections over the past few years. The efficacy of the application of fiber reinforced composites as a strengthening technique for RC structures has been studied and experimented in recent years due to their outstanding advantages over steel including their light weight, high corrosion resistance, high strength and, last but not least, their ease of application. FRP materials with various fiber types of carbon, glass or aramid provide a cost-effective and adaptable strengthening and retrofitting approach for reinforced concrete structures.

This study is an investigation into the utilisation of carbon FRP composites in retrofitting defected beam-column connections which had no transverse shear reinforcement in the joint region. Moreover, the effects of increasing the number of CFRP sheets in the joint region and also the consequences of modifying the joint cross section from rectangular to circular on the overall behavior of the joint were considered in this study.

1.2. Rationale

One of the most desirable and efficient systems used in construction to increase the energy dissipation capacity and lateral stability of structures under seismic loadings is the Reinforced Concrete Moment Resisting Frame (RCMRF). The basic approach in the design phase of an RCMRF is the “Strong Column, Weak Beam” concept; where elastic behavior of joints in the frames are of great importance (Kim et al. 2009). Beam-column connections are considered as one of the weakest components of RCMRFs when subjected to seismic lateral loadings. The overall strength, stiffness and ductility of structures are considerably dependent on the
performance of beam-column connections and the adjacent members. Beam-column joints bear horizontal and vertical shear forces whose values on the cross section passing through the joints are much larger than those in the vicinity of the joint. Consequently, these connections are more prone to shear failure which is usually a brittle one. In order to ensure a ductile behavior for RCMRFs, brittle failure of the joint needs to be avoided through proper design and strengthening. Brittle failure modes in beam-column connections threaten the integrity of the whole structure and in most cases result in the total collapse of the frame. Evidence from earthquakes like the 1999 Kocaeli, Turkey earthquake, indicated that several structure failures were attributed to the brittle failure of beam-column connections (Ghobarah & Said 2002).

Several methods have been proposed for strengthening purposes of the beam-column joints. The most common techniques employed to upgrade the shear capacity and ductility of RC joints are concrete jackets, steel jackets and, most recently, FRP composites; each with their own merits. Nonetheless, FRP composites have shown to be an efficient strengthening material due to their characteristics. One of the most important aspects of the retrofitting and rehabilitation of beam-column connections is the issue of providing sufficient confinement to the joint. FRP composites, due to the method of the application of FRP composites to the surface of concrete with the use of Epoxy, trigger enhanced confinement of concrete in the joint region. Nevertheless, the presence of sharp edges in the vicinity of the joint causes stress concentration in the corners which, in effect, results in early debonding of FRP material and as a result, failure of joint before reaching its ultimate capacity. This issue was also addressed in this study. In the proposed technique in this study, measures were taken to modify the cross-section near the joint area to eliminate sharp corners and prevent the concentration of stress in the vicinity of the beam-column connection.

1.3. Objectives

This study aimed to investigate the efficacy and also the practicality of using CFRP composites to strengthen beam-column connections under cyclic loading in order to increase the shear capacity of the joint and also to prevent brittle failure and enhance connection ductility. The detailed objectives of the study were as follows:
• Examining the general behavior of CFRP strengthened specimens under displacement control cyclic loading compared to the control Specimen SP1
• Examining the general behavior of CFRP strengthened specimen, SP2 compared to Specimen SP3 which was reinforced with six times the number of CFRP layers used for Specimen SP2
• Scrutinising failure modes and ductility of CFRP strengthened specimens compared to those of the control specimen
• Analysing the ultimate strength of the three specimens to deduce the effects of external CFRP reinforcement of the joint on specimen capacity
• Investigating the effects of joints’ external reinforcement on displacements in the vicinity of the beam-column connection
• Monitoring the influence of joint cross-section modifications on CFRP performance
• Exploring the effects of external CFRP reinforcement on strains in steel reinforcements in the joint vicinity

At the end, all results were analysed and compared to those reported by other scholars in the literature and eventually deductions were made.

1.4. Thesis Outline

This thesis consists of five chapters. The chapters following the Introduction Chapter and their contents are explained below:

Chapter 2: Literature Review examines the characteristics of beam-column connections as well as those of FRP composites. In addition, relevant studies on beam-column connections, their performance and also strengthening and rehabilitation techniques used for beam-column joints are presented and compared. Chapter 3: Research Design and Methodology covers the methodology behind specimen preparation, material properties, experimental set up, testing procedures and data collection. Chapter 4: Results presents the results recorded from the cyclic loading test on each of the specimens as well as the control tests on concrete and steel rebars. In Chapter 5: Discussion, the results of the tests on the three specimens are analysed, compared to one another and discussed. Moreover, the results are then compared to those presented in the
literature review by other scholars. In the end, according to the presented analyses, conclusions are made on the outcome of the experiment.
Chapter 2 : Literature Review

2.1. Introduction

This chapter presents the characteristics, strengths and weaknesses of beam-column connections. As described in the Introduction chapter, this study aims to investigate the efficacy of applying CFRP sheets to beam-column connections as a means to increase the load carrying capacity of the joints. According to Hanson et al. (1967), the Portland Cement Association took the initiative in performing tests on beam-column joints in early 1960s. Shrestha et al. (2009) and Pimanmas & Chaimahawan (2012) also mention that for the past four decades, extensive research has been done on the behavior of joints under different seismic loading conditions through experimental and analytical studies and some of the findings have brought about revisions to various international codes of practice. In this chapter, pertinent studies have been reviewed in order to provide better understanding of the parameters affecting the shear strength of T-connections, various strengthening and retrofitting methods for weak or damaged T-connections and also different testing procedures and loading conditions for better evaluation of the capacity of the beam-column connections (Wang et al. 2012, Alva et al. 2007, Hadi 2011, Li et al. 1999, Shrestha et al. 2009, Mukherjee et al. 2004).

2.2. Beam-Column Connections

American Concrete Institute 352R-02 describes a beam-column connection as “The joint plus the columns, beams and slabs adjacent to the joint”. Hadi (2011) describes a beam-column connection as “a common point of intersection of columns and beams which provides resistance to applied external loads due to the bending moment encountered at the joint”. Beam-column connections are considered as one of the weakest components of RCMRFs (Reinforced Concrete Moment Resisting Frames) when subjected to seismic lateral loadings. RCMRFs could experience various levels of damage during strong earthquakes. These damages either stem from faulty designs or poor workmanship. Therefore, the design and construction of beam–column connections are key elements in the seismic strengthening and retrofitting of reinforced concrete frame buildings (Vatani-Oskouei, 2010). In earthquake-prone regions, the joints must be designed to allow the dissipation of large amounts of energy into the neighboring elements without a significant loss of strength and ductility. Connection failure has been treated as a
crucial topic due to the fact that failure at beam-column joints often results in the failure of the whole structure (Tehrani, 2008). The most desirable type of failure in beam-column connections is the formation of plastic hinges in the beam away from the joint. In this type of failure, no critical damage is rendered to the joints and, therefore, the energy dissipation capacity of the structure, which stems from joint ductility, will not vary drastically. Nonetheless, if plastic hinges form inside or in the vicinity of the joint and if severe damage is caused within the joint panel, the failure of the beam-column connection can threaten the integrity of the whole structure (Kim et al, 2009). Due to the importance of this topic a lot of research has been done in order to find the causes of joint failure and also to propose proper solutions to the issue. (Tehrani 2008, Paulay 1989, Kim et al. 2009, Wang et al. 2012, Alva et al. 2007, Hadi 2011, Li et al. 1999, Shrestha et al. 2009, Mukherjee et al. 2004).

According to Paulay (1989), there are two basic mechanisms that lead to failure within the joint region: 1. failure within the joint caused either by the bond between reinforcements and concrete or by shear failure of the joint or a combination of the two, and 2. yielding of the beam reinforcements adjacent to the joint. In the year 2004, another study investigating the influence of certain parameters affecting the behavior of reinforced concrete beam-column connections under cyclic loading was carried out by Alva et al. (2007) at the University of Sao Paulo. The main parameters analyzed and studied by Alva et al. (2007) were joint shear reinforcements and concrete compressive strength. The results of the tests in that study showed that although both concrete compressive strength and the amount of joint shear reinforcements contribute to the ultimate strength of the joint, the former plays a bigger role and therefore is of greater importance. Alva et al. (2007) experimented on four different specimens and compared their results. Two of the specimens were cast with the same strength concrete; however, one was designed with more joint stirrups than the other. The results of the tests on these two specimens indicated that the presence of additional shear reinforcements in the joint area effectively increased the capacity of the beam-column connection under cyclic loading. In the next stage of the test, the other two specimens were tested on and the results were analyzed. The compressive strength of the concrete used to cast one of the specimens was 44.18 MPa whereas the compressive strength of the concrete used for the other specimen was 25.91 MPa. Despite the fact that the latter had twice as many stirrups in the joint as the former, it still showed a 40%
lower shear strength. Consequently, the conclusion was made by Alva et al. (2007) that the compressive strength of concrete is far more important than the number of shear reinforcements in the joint in determining the shear capacity of a reinforced concrete beam-column connection.

The conclusions made by Alva et al. (2007) were supported by an analytical study carried out by Kim et al. (2007) which indicated that the compressive strength of concrete was the most common governing parameter on joint shear behavior. In their study, Kim et al. (2007) constructed an extensive database of reinforced concrete beam-column connection test specimens which had exhibited joint shear failure. The main objective of the study was identifying the key parameters for the joint shear behavior of reinforced concrete beam-column connections. The parameters examined by Kim et al. (2007) were compressive strength of concrete, joint panel geometry, reinforcement confinement, column axial load and reinforcement bond condition. The results of the investigations indicated that an increase in the compressive strength of concrete brought about an improvement of joint shear resistance which stems from force transfer to the joint panel and also from the bonding between the steel reinforcements and the surrounding concrete; consequently, it was deduced that the compressive strength of concrete is of grave importance in determining the ultimate strength of a reinforced concrete beam-column connection.

Supaviriyakit & Pimanmas (2008) also explored different parameters influencing the cyclic behavior of substandard interior beam–column connections. The main parameters investigated by Supaviriyakit & Pimanmas (2008) were column size and substantial horizontal joint reinforcements. The study indicated that as the column size increased, joint shear stress decreased and the bond demand for column reinforcing bar lap splice was reduced.

As to substantial horizontal joint reinforcements, Supaviriyakit & Pimanmas (2008) stated that the ACI minimum joint shear reinforcement may not be adequate for moderately ductile joint performance since shear failure of joints were observed during the experiment in cases where substantial horizontal joint reinforcement was provided.

Park & Mosalam (2012) supported the findings by Supaviriyakit & Pimanmas (2008) in their study on Parameters for shear strength prediction of exterior beam–column joints without transverse reinforcement. The study showed that the shear strength of unreinforced exterior
joints reduced when the joint aspect ratio, i.e. the ratio of the beam to column cross-section heights increased. Park & Mosalam (2012) also stated that the shear strengths of unreinforced exterior joints were not systematically affected by the compressive column axial loads smaller than $P \leq 0.2 f'_{c} A_g$.

2.3. Strengthening RC Beam-Column Connections

Due to the importance of reinforced concrete beam column connections numerous experimental tests and analytical studies have been performed since the 1960s to investigate the parameters affecting the ultimate strength of beam-column connections as well as new ways of strengthening these connections against seismic loadings. The reason why these connections are such critical subjects is that brittle failure of beam-column joints could result in catastrophic collapses of structures (Pimanmas et al. 2010). Two general approaches are usually considered for the rehabilitation of buildings damaged by seismic motions. The first approach involves the modification of the entire structural system, whereas the second approach deals with the modification of some components of the structural and non-structural system (Vatani-Oskoue 2010). Over the years many different methods for strengthening and retrofitting beam-column connections have been proposed such as concrete jacketing, steel jacketing and fiber reinforced polymer sheets.

2.3.1. Concrete Jackets

Concrete jacketing is one of the common techniques used for seismic strengthening of beam-column connections. Alcocer et al. (1993) studied concrete jackets as a way to rehabilitate reinforced concrete frame connections. Although concrete jacketing is reportedly an effective way to retrofit damaged beam-column connections it has its own disadvantages. One of its demerits is that it reduces usable floor space as concrete jackets are somehow bulky and the joints might become architecturally unacceptable after rehabilitation. Tsonos (2007) carried out a study on the effectiveness of concrete jacketing and FRP jacketing as pre-earthquake and also post-earthquake strengthening techniques for beam-column joints and the results of the tests indicated that, despite its disadvantages, concrete jacketing still remained an effective method for retrofitting damaged and deficient joints specifically in the post-earthquake scheme. Tsonos (2007) reported that in the case of pre-earthquake retrofitting both concrete jackets and FRP
composites demonstrated equal efficiency. However, in the case of post-earthquake retrofitting, concrete jacketing proved to be slightly more effective than FRP composites.

### 2.3.2. Steel Jackets

Another widely utilized method for retrofitting beam-column connections is using steel jackets and steel straps. Hadi (2011) carried out a study on the effectiveness of using steel straps to rehabilitate a damaged reinforced concrete T-connection. The specimen was initially tested to failure under static loading and was then rehabilitated using Epoxy (EP40 and EP10), galvanised steel and steel straps. Once rehabilitated, the specimen was tested again under the same loading condition. The results of the tests before and after rehabilitation were then analyzed and compared by Hadi (2011) to determine the performance level of the rehabilitated specimen. According to results reported by Hadi (2011), the rehabilitated specimen exhibited a lower yield load but a 13.67% higher ultimate load compared to the original standard specimen. There was, however, no increase in the rotational capacity of the joint in the rehabilitated specimen compared to the original one. Hadi (2011) also reported a brittle failure for the rehabilitated specimen while the original specimen underwent a ductile failure mode. Overall, despite the fact that the steel reinforcements had yielded in the initial test on the original specimen, the rehabilitated specimen exhibited satisfactory performance in carrying loads before eventually failing. However, there are a few disadvantages to this technique also. Steel jackets and steel straps are vulnerable against corrosive materials which could affect their performance after some time. Moreover, the relatively difficult application of steel jackets and steel straps is considered as one the shortcomings of this method.

### 2.3.3. FRP Composites

#### 2.3.3.1. Introduction

Fiber Reinforced Polymers are increasingly being used to strengthen concrete, masonry and timber. Different types of FRP materials including glass, carbon and aramid are used to strengthen or repair structures by offering cost-effectiveness, adaptability, high stiffness to weight ratio, and strong corrosion resistance (Li et al. 1999). Externally bonded fiber reinforced polymer composites are also used to strengthen RC beam–column connections. Experiments demonstrate that the application of FRP composites can enhance shear capacity, anchorage
capacity and can also relocate the formation of plastic hinges further along the beam and away from the joint (Shrestha et al. 2009). Several studies have been conducted on the efficiency of using FRP composites as a method of rehabilitation for damaged or defected beam-column connections (Mahmoud et al. 2012, Lee et al. 2010, Zhoudao et al. 2011, Gergely et al. 2000, Dalalbashi et al. 2012). In this chapter, these studies and their methodologies have been discussed and the results have been analysed and compared in order to provide better understanding of the rationale used in this experiment.

2.3.3.2. Types of Fiber Reinforced Polymer Composites

As explained in the Introduction section, different types of fiber reinforced polymers (GFRPs, CFRPs and AFRPs) are used in structural rehabilitation (Li et al. 1999). The reported results of experiments carried out by Mukherjee et al. (2005) on the behavior of reinforced concrete beam-column joints rehabilitated by GFRP and CFRP sheets and plates indicate that both composites enhanced the performances of joints under cyclic excitation by increasing the ultimate strength and ultimate deflection and also by heightening the energy dissipation capacity of connections. However, the comparison between specimens strengthened with GFRP and CFRP composites suggested that carbon fiber reinforced specimens exhibited higher energy dissipation capacity than glass fiber reinforced specimens which Mukherjee et al. (2004) attributed to higher stiffness of carbon fiber sheets. The specimens reinforced with GFRP sheets, on the other hand, had a higher displacement at yield than CFRP strengthened specimens.

Krishna & Ravindra (2012) also carried out a series of experiments in order to record the feasibility of using carbon FRP and glass FRP sheets to rehabilitate moderately damaged reinforced concrete interior beam-column connections. In their testing scheme, a total of eight specimens were tested to failure under static loading. Four of the damaged specimens were then retrofitted using GFRP sheets and the remaining four were strengthened by applying CFRP sheets. The results of their study indicated that both carbon and glass fiber reinforced polymers significantly increased the ultimate strength and stiffness of the specimens. The findings by Ramakrishna & Ravindra (2012) were similar to those reported by Mukherjee et al. (2004) in terms of displacement as the specimens reinforced with GFRP had higher displacement at yield than those reinforced with CFRR. Nevertheless, Ramakrishna & Ravindra (2012) reported that
the GFRP strengthened specimens demonstrated higher stiffness than CFRP ones which is in conflict with findings by Mukherjee et al. (2004). As to ultimate load capacities, both Ramakrishna & Ravindra (2012) and Mukherjee et al. (2004) observed that CFRP sheets rendered higher yield points for damaged specimens than GFRP sheets.

In a qualitative comparison of different types of fiber reinforced polymers, Meier (1995) states that due to their high tensile strength, fatigue and alkaline resistance and also because of their stiffness and long term durability, carbon fibers, at times, offer more desirable traits as structural reinforcements than glass and aramid fiber reinforced polymers. Nonetheless, Antonopoulos & Triantafillou (2003) maintain that in their experiment GFRP reinforcements showed to be more effective than CFRP reinforcements as specimens strengthened with glass fiber composites had higher energy dissipation capacities than those strengthened with carbon fiber composites. The authors also suggested that more tests be done on this subject as their conclusions were based on one test only.

A study by Li et al. (1999) indicated that combining different types of fiber reinforced polymer composites into a hybrid FRP could result in a stronger composite which would provide better reinforcement for defected or damaged beam-column connections. The FRP composite used by Li et al. (1999) was a hybrid of E-glass woven roving (WR/600 g/m²), chopped strand mat (CSM-300 g/m²), carbon cloth (plain weave-200 g/m²) and glass fiber tape (GFT-250 g/m²). According to Li et al. (1999), the reason for choosing this hybrid of composites was that GFT provided excellent confinement and enhanced the integrity of the structure and that WR and carbon cloth had the ability to provide multi-directional reinforcement and played the main role in the strengthening of the joints in the study. The results showed a 45% increase in stiffness of the specimen with FRP reinforcements in service load level compared to specimens with no FRP reinforcements. The ultimate load level curves also showed that the FRP reinforced specimen had a 30% higher ultimate load capacity than did the other Non-FRP reinforced samples. Based on the results of the tests, Li et al. (1999) concluded that not only did the application of hybrid FRP composites triggered significant increase in stiffness and load carrying capacity of the specimens but it also contributed to good bonding and no spalling of concrete.
2.3.3.3. External Reinforcement of beam-column connections

Due to the sensitive nature and critical role of beam-column connections in structures, the application of fiber reinforced polymers as a retrofitting technique has increased in recent years because of their quick and easy application and short construction time (Ghobarah & Said 2002). Although fiber reinforced composites have proven to be effective in increasing the strength and ductility of joints, the search for the optimum configuration of FRP sheets in the vicinity of beam-column connections still continues. According to experiment outcomes reported by Gergely et al. (1998), Pantelides et al. (1999), Spadea et al. (1998), Dalalbashi et al. (2012) and Shrestha et al. (2009), the two main issues in the external reinforcement of beam-column connections with FRP composites are providing sufficient confinement to the joint and also preventing the debonding of FRP sheets which results in crack formation and propagation. Mahini & Ronagh (2010) explain that FRP strengthened beam-column connections might exhibit one of the following major failure modes:

1. De-bonding of FRP followed by crack propagation

2. Yielding of reinforcing steel in tension followed by the rupture of FRP (FRP rupture).

3. Yielding of reinforcing steel in tension followed by concrete crushing (tension failure).


Ghobarah & Said (2002) further state that if necessary measures are taken to provide sufficient confinement to the joint area, most failure modes could be avoided or delayed.

2.3.3.3.1. FRP Strengthened Beam-column Joints

In an investigation into the behavior of beam-column connections under seismic loading, Gergely et al. (2000) carried out a study on shear strengthening of reinforced concrete T-connections using carbon FRP composites. The results of the tests suggested that carbon FRP composites greatly improved the shear capacity of the joints in the study. Moreover, Gergely et al. (2000) reported that the application of CFRP composites improved the specimens’ overall damage control and also provided the joints with enough lateral confinement to support dead
load after reaching failure. The specimens used by Gergely et al. (2000) were all designed so that there were no transverse reinforcements in the beam or the beam-column joint area in order to make it easier to identify how much each of the resisting components (concrete, reinforcing steel and CFRP sheets) contributed to the shear capacity of the joint. Another study performed by Lee et al. (2010) on RC beam-column connections strengthened with carbon fiber reinforced polymer supported the findings of Gergeley et al. (2000). The outcome of the tests by Lee et al. (2010) implied that joints reinforced with CFRP showed a higher level of stiffness, strength and energy dissipation capacity. Similar to the methodology used by Gergeley et al. (2000), Lee et al. (2010) also did not use any transverse reinforcements in the joint area in order to observe the performance of fiber reinforced polymer composites. The same approach was used by El-Amoury and Ghobarah (2001) in their experiment on seismic rehabilitation of beam-column joints with GFRP sheets. The authors state that specimens with no shear reinforcement in the joint and with inadequate anchorage for the beam steel reinforcement showed a brittle joint failure; however, specimens strengthened with GFRP sheets demonstrated no brittle failure and had much higher energy dissipation capacity. The strengthened specimens exhibited a 52% higher ultimate load capacity than that of the control specimen with no GFRP reinforcement. In a similar study on using FRP as a method of retrofitting reinforced concrete joints, Dalalbashi et al. (2012) complements the findings by Lee et al. (2010) and Gergeley et al. (2000). Niroomandi et al. (2010) investigated an eight-story frame strengthened and retrofitted with web-bonded CFRP. The reported results by Niroomandi et al. (2010) suggested that the web-bonded FRP retrofitting at joints resulted in a 40% increase in the lateral ultimate strength of the original reinforced concrete frame. The over-strength of the frame was also enhanced by 66% thanks to the presence of CFRP. The ductility of the RC frame was also enhanced significantly. Moreover, Niroomandi et al. (2010) observed that due to the increased strength capacity and improved ductility, both caused by FRP retrofitting of the joints, the seismic behavior factor of the frame was substantially increased by over 100%, resulting in an over 50% reduction in the seismic base shear.

Vatani-Oskouei (2010) also looked into the rehabilitation of damaged reinforced concrete beam-column connections by the application of CFRP sheets. Two full scale exterior beam-column connections were tested to failure and then retrofitted with CFRP sheets to explore the ability of
fiber composites to restore ultimate shear strength of the joints. The test results indicated that the application of CFRP sheet to the damaged joints increased the system strength, stiffness, damping ratios, and energy dissipation capacities. Moreover, thanks to CFRP reinforcements, plastic hinges were relocated in the beam and further away from the joint which in turn increased the ductility of the rehabilitated connection. Vatani-Oskouei (2010) reported a 97% increase in the ductility of the first retrofitted specimen and a 50% increase in the ductility of the second one. In addition, the load carrying capacities of the first and second specimens increased by 17% and 32%, respectively.

2.3.3.3.2. FRP Reinforcement Configurations

While there seems to be a unanimous verdict on the efficacy of FRP composites in their application as reinforcing materials for beam-column connections, the search for an optimum configuration to trigger higher load carrying capacity, ductility and energy absorption capacity still continues. In an investigation into different FRP configurations, Dalalbashi et al. (2012) focused on the relocation of plastic hinges from the column face towards the beam in order to prevent brittle joint failure by experimenting on three reinforced concrete beam-column connections. Dalalbashi et al. (2012) used a different FRP configuration on each of the three specimens in order to identify which configuration was more effective in increasing the ultimate strength of the joint as well as relocating plastic hinges away from the column face. The results indicated that in all three configurations, there was a significant increase in the ultimate strength of the joint. However, only two configurations demonstrated the ability to move plastic hinges further towards the beam. Dalalbashi et al. (2012) also concluded that the configuration of the FRP composites and the thickness of the FRP sheets used in the process played a big role in the final results. Figure 2.1 shows the three configurations used by Dalalbashi et al. (2012).
In the first case, the FRP sheets were applied in an L-shape and the applied sheets were then wrapped in order to prevent early spalling of composites near the joint. In the second case, FRP sheets were applied to the top and bottom of the beam but no external reinforcement was applied to the column. The third configuration was the same design as the second one with only one modification; FRP laminates were inserted into the concrete for a length equal to the concrete cover in order to prevent debonding of FRP sheets. Dalalbashi et al. (2012) reported that all three configurations increased the ultimate strength of the joint with the highest value in the third design where FRP laminates were inserted into the concrete cover. However, it was noted that only the first and third configurations were capable of relocating plastic hinges away from the joint. Consequently, Dalalbashi et al. (2012) concluded that the configuration of FRP reinforcement, the thickness of FRP sheets and also the number of layers of composites applied to the joint are of great consequence in determining the ultimate strength of the beam-column connection.

There have been a number of other scholars and researchers who investigated different FRP configurations on RC beam-column joints (Dalalbashi et al. 2012, Shrestha et al. 2009, Mahmoud et al. 2013, Mukherjee et al. 2005). In a similar study, Shrestha et al. (2009) investigated the efficacy of two carbon FRP strengthening schemes on T-connections. In their study, the primary objective of the experiment was to observe the behavior of the FRP strengthened area, crack formation and propagation and also failure modes of strengthened connections rather than focusing on overall behavior of the connections such as strength, ductility and energy absorption capacity. For this purpose, carbon FRP strips were chosen as
opposed to carbon FRP sheets to make it easier to monitor the areas in the vicinity of the composites in order to observe failure modes, crack formations and de-bonding of the CFRP strips from the concrete. The analysis of the results by Shrestha et al. (2009) indicated that the application of CFRP strips were very effective in preventing the formation of diagonal shear cracks in the joints. Although Shrestha et al. (2009) took measures to prevent complete debonding of FRP strips by using FRP wraps on both ends of the strips, similar to the experiments by Dalalbashi et al. (2012), the tests demonstrated that local de-bonding of the strips still occurred which caused specimens to fail before the full capacity of the CFRP strips could be reached. The two FRP configurations used by Shrestha et al. (2009) are shown in Figure 2.2.

![Figure 2-2 a) Column strip scheme b) Beam strip scheme (Shrestha et al. 2009)](image)

Observations reported by Shrestha et al. (2009) suggest that both column strip scheme and beam strip scheme increased the ultimate strength and the deflection of the beam-column connection. The beam strip scheme, however, proved to be more effective in hindering crack formation and crack propagation. Shrestha et al. (2009) and Dalalbashi et al. (2012) both reported that the configurations providing more confinement and support to the beam face of the joint tend to be more effective in enhancing the ultimate strength of the joint. Antonopoulos & Triantafillou (2003) also reported similar results when they investigated the distribution of FRP between beam and column amongst other parameters in their study. The readings indicated that the specimens with beam FRP reinforcement showed a strength increase of 70% while the figure for the
specimen with column FRP reinforcement was 50%. Respectively, the reported increments for energy dissipation were 40% and 20%.

Amongst other variables examined by Antonopoulos & Triantafillou (2003) were column axial load, FRP sheets versus strips and Glass FRP versus Carbon FRP. According to the authors, the higher the axial load on the column was, the higher the shear strength of the FRP-reinforced joint would be. It was also reported that FRP sheets transferred forces more evenly than FRP strips and that specimens reinforced with FRP sheets performed better than those reinforced with FRP strips. As to the comparison between the effectiveness of GFRP and CFRP composites, Antonopoulos & Triantafillou (2003) stated that in terms of strength both composites proved to be significantly effective. However, GFRP strengthened specimens exhibited higher energy dissipation capacity than those reinforced with CFRP sheets. Antonopoulos & Triantafillou (2003) also investigated different FRP configurations of single and multiple layers of FRP sheets and strips on the beam and column faces of the joints. It was reported that configurations with two layers of FRP sheets exhibited shear strength and energy dissipation capacities of 65% more than single layer configurations.

In their study, Parvin & Wu (2008) looked into the behavior of three beam–column connections that were strengthened with four layers of CFRP with various ply angle configurations. Parvin & Wu (2008) used a different configuration on each of the specimens; the first specimen was wrapped with four layers of CFRP with ply angle configurations of $0^\circ/90^\circ/0^\circ/90^\circ$ respectively, the second specimen was wrapped with the same number of layers and ply angle configuration of $0^\circ/90^\circ/-45^\circ/+45^\circ$ and the third specimen was wrapped with four layers of CFRP and ply angles of $-45^\circ/+45^\circ/-45^\circ/+45^\circ$. In order to avoid stress concentration at sharp edges and also to provide more confinement to the beam-column connection, the edges of the beam and column were rounded with a radius of 0.5 in (12.7 mm) before the application of CFRP layers; the same approach was utilised in this study but only to the edges of the beam in the vicinity of the joint as another technique was used to avoid stress concentration in the column area, both techniques will be described in some detail in the next chapter. Parvin & Wu (2008) reported that the presence of CFRP wraps was very effective in retrofitting the existing beam-column joints and that the wrapping ply angle configuration was probably a relatively important factor in the shear resistance of the beam-column connection. Amongst the three configurations used by Parvin &
Wu (2008), wrap ply angle stacking sequence of -45°/+45°/-45°/+45° offered the most resistance against brittle shear failure of the joint and also provided the highest energy dissipation capacity when subjected to combined axial and cyclic loads.

Various FRP strengthening configurations were also investigated by Mahmoud et al. (2013) who studied different strengthening schemes on beam-column joint with different defects. Mahmoud et al. (2013) studied two FRP configurations on beam-column connections with no stirrups in the joint area and two configurations for joints with insufficient bond length for the beam’s main steel reinforcements. The two configurations for joints with no stirrups are shown in Figure 2.3. Mahmoud et al. (2013) reported that both schemes increased the ultimate capacity of the joints. However, the diagonal application of FRP strips proved to be the better choice in the absence of stirrups within the joint area.

Figure 2-3 FRP configurations for beam-column joints with no strirrups in the joint (Mahmoud et al. 2013)

For the second case of defected joints, Mahmoud et al. (2013) employed the two FRP configurations which are illustrated in Figure 2.4. The L-shaped application of FRP sheets reportedly triggered better results in the case of insufficient bond length for beam reinforcements. As can be seen in Figure 2.4, the second scheme provided better confinement to the beam face of the joint than the first scheme.
2.3.3.3. Confinement of beam-column connection

In most of the reviewed literature, it was concluded that using fiber reinforced polymer composites as a method of rehabilitation for damaged or defected reinforced concrete beam-column connections is an effective retrofitting strategy and the final results indicated that by applying FRP sheets to the joints, the ultimate load carrying capacity, energy dissipation capacity and ductility of the joints could be enhanced. Nonetheless, the configuration of FRP sheets used in the retrofitting scheme is a key factor in determining the extent to which composites contribute to the strengthening of beam-column connections (Ghobarah & Said 2002, El-Amoury & Ghobarah 2002 and Alsayed et al. 2010). Moreover, the amount of confinement to the joints as a result of the presence of FRP sheets is also another factor affecting the formation and propagation of cracks in the vicinity of the connection. The results of an earlier study by Mirmiran et al. (1998) indicate that the confinement effectiveness of FRP jackets in concrete columns and beams is influenced by several factors including concrete strength, types of fibers and resin, fiber volume and orientation, jacket thickness, shape of cross section, length-to-diameter (slenderness) ratio of the column, and the interfacial bond between the concrete core and the jacket. Mirmiran et al. (1998) also reported that the shape of the column cross sections and the corner radius ratio played an important role in providing sufficient confinement to increase the strength of columns. In another approach to examining the effects of column shape on the performance of carbon fiber reinforced polymer wraps, Yang et al. (2004) reported that a
smaller corner radius can significantly reduce the ultimate strength of the FRP laminate due to stress concentration around the corner area. The collected data by Yang et al. (2004) showed an increase in the stress concentration factor when the corner radius was reduced.

In their study on the seismic response of FRP upgraded RC beam-column connections, Alsayed et al. (2010) experimented on four as-built specimens; two of which were reinforced with CFRP sheets in the joint area. In one of the two FRP reinforced specimens, Alsayed et al. (2010) eliminated the possibility of FRP debonding through mechanical anchorage. The reported results indicated that both schemes effectively improved shear strength and deformation capacity of specimens; however, debonding of FRP sheets in the specimen without mechanical anchorage caused cracks to form under the composites which eventually resulted in joint failure under seismic loading. In the second specimen where the possibility of FRP debonding was eliminated, the joint was made so strong that failure was directed to the adjacent beam and column. In a similar study, El-Amoury & Ghobarah (2002) tested two specimens. One of the specimens was simply reinforced with GFRP sheets while the other was also reinforced with U-shaped steel plates to strengthen GFRP sheets against debonding from the concrete surface. The results showed a higher ultimate load capacity for the second specimen which was reinforced against FRP debonding. However, Alsayed et al. (2010), El-Amoury & Ghobarah (2002) and Ghobarah & Said (2002) reported that due to joint section properties resulting from rectangular columns, only limited confinement could be provided to the joint which increased the possibility of bulging of the FRP wraps. Bulging allows cracks to form and widen under the FRP sheets.

After reviewing the previously discussed literature on more effective FRP configurations for external reinforcement of beam-column connections, the initial configuration to be used in this study was chosen and a few modifications were made later on in the study to optimise the contribution of FRP sheets to the ultimate strength and the overall behavior of joints under cyclic loading.
2.4. Conclusion

Reinforced concrete beam–column (RCBC) joints are one of the most complex, least studied and critical structural components of a building or bridge structure that is subject to seismic loading. Experimental investigations have determined that stiffness and strength reduction of beam-column joints results in stiffness and strength reduction of the whole structure and the failure of reinforced concrete beam-column joints may even lead to catastrophic collapse of the entire structure. Inadequate transverse reinforcement in the joint and weak-column/strong-beam design are the main reasons for the observed joint shear failures during recent earthquakes. Joint shear failures may result in non-ductile performance of reinforced concrete moment-resisting frames. Many existing structures were designed and built before the development of current seismic codes, or on the basis of earlier codes before ductile reinforcement detailing was required. The first design guidelines for reinforced concrete beam-column joints were published in 1976 in the U.S. (ACI 352R-76) and in 1982 in New Zealand (NZS 3101:1982). For these older structures, there is usually the problem of insufficient shear reinforcement in the joint area and also the issue of low ductility. It is important that the system has enough ductility to allow loads to redistribute and prevent brittle failure. Older designs often do not have proper reinforcement details needed to ensure ductile behavior (Wang et al. 2012, Vatani-Oskouei 2010, Tehrani 2008, Kim et al. 2009, Paulay 1989, Kim et al. 2009).

In order to strengthen weak beam-column connections and also to rehabilitate damaged joints, many different methods for strengthening and retrofitting beam-column connections have been proposed over the years such as concrete jacketing, steel jacketing and fiber reinforced polymer sheets. Due to their structural characteristics, FRP composites are often favored over other methods of rehabilitation. It has been acknowledged that externally bonded fiber reinforced polymer (FRP) composites can effectively strengthen reinforced concrete beam-column connections. Both exterior and interior connections have been tested with externally bonded FRP to enhance their shear capacity and also to relocate plastic hinges further along the beam away from the joint (Shrestha et al. 2009, Mahmoud et al. 2012, Lee et al. 2010, Alcocer et al. 1993).

A number of other scholars have also reported increased shear capacity, enhanced ductility, higher energy dissipation capacity, increased stiffness, higher seismic behavior factor and

Amongst different types of fiber reinforced polymer composites, Carbon FRP and Glass FRP are most commonly used for the purpose of external retrofitting of beam-column connections. Studies show that both CFRP and GFRP are effective reinforcing materials and can be utilised in the rehabilitation of damaged or defected joints. There are, however, slight differences in the performances of specimens reinforced with CFRP and GFRP. Mukherjee et al. (2004) reported that in their experiments, specimens reinforced with CFRP had a higher level of stiffness while specimens reinforced with GFRP demonstrated higher displacement. Nonetheless, more tests need to be done to carry out a better comparison between the two composites.

Although FRP composites have proven to be effective in strengthening beam-column connections, the search for the optimum configuration, methods of application and also techniques for the prevention of debonding of FRP composites still continues. After examining various FRP configurations on RC beam-column connections Shrestha et al. (2009) and Dalalbashi et al. (2012) reported that fiber arrangements providing more confinement and support to the beam face of the joint tend to be more effective in enhancing the ultimate strength of the joint. Antonopoulos & Triantafillou (2003) stated that applying two layers of FRP increased the shear strength and energy dissipation capacities of the specimen 65% more than single layer configurations.

Another important factor in the performance of FRP jackets is the confinement of the joint area. As Alsayed et al. (2010), El-Amoury & Ghobarah (2002) and Ghobarah & Said (2002) reported, in the case of rectangular columns and due to the presence of sharp edges the amount of confinement provided to the joint was limited which resulted in bulging of the FRP sheets which in turn allowed the formation and propagation of cracks. In order to provide sufficient confinement to the beam-column connection, Parvin & Wu (2008) took measures to grind the edges of the beam and column with a radius of 0.5 in (12.7 mm) before the application of CFRP layers.
After reviewing the previously discussed literature on FRP composites, joint confinement and the most efficient FRP configuration for external reinforcement of beam-column connections, the initial configuration to be used in this study was chosen and a few modifications were made later on in the study to optimise the contribution of FRP sheets to the ultimate strength and the overall behavior of joints under cyclic loading. The specimen preparation processes along with material properties selected for this study are explained fully and in detail in the next chapter.
Chapter 3 : Research Design and Methodology

3.1. Introduction

This chapter of the thesis focuses on the research design and methodology adopted in this study to achieve the aims and objectives mentioned above in Chapter 1. The afore-mentioned objectives of this study include:

- Examining the effects of CFRP strengthening on the overall behavior of a beam-column connection under cyclic loading
- Comparing the ultimate strength of CFRP strengthened specimens and the control specimen
- Analysing the influence of joint cross-section modifications from rectangular to circular on the overall performance of the composites
- Comparing the performance of specimens retrofitted with one and multiple layers of CFRP in the joint region
- Exploring the effect of external CFRP reinforcement on strains in steel reinforcements in the joint vicinity

Once relevant literature on the topic of this study was reviewed, the research design was outlined accordingly and optimum reinforcing techniques and testing procedures were decided on. The two main parts of the literature which influenced the research design were confinement of beam-column connections and FRP reinforcement configurations. This chapter also provides a step by step narration of the experiment process from specimen preparation and material properties to test set-up and analysis of results for each specimen.

3.2. Specimen characteristics

A total of three specimens were designed and cast in the SMART lab at the University of Wollongong. The details of the dimensions of the specimens as well as reinforcement details are shown in Figure 3-1. The column-beam T-section was designed according to AS3600-2009 to detail required quantities of reinforcements.
Figure 3-1 Specimen Dimensions and Reinforcement Details
In order to ensure that the behavior of the beam-column connections would be conspicuous under cyclic loading and also to be able to observe the influence of the CFRP wraps around the joint area, no transverse shear reinforcement was used in the joint region and steel bars and stirrups were only used as longitudinal and shear reinforcements in the column and the beam. One of the specimens, SP1, was used as the reference specimen and therefore required no FRP strengthening. The other two specimens, SP2 and SP3, were initially cast with the exact same dimensions and steel reinforcements as the reference specimen but were then modified by attaching separately cast concrete in order to transform the joint cross-section into a circular one to provide additional confinement to the beam-column connection. These two specimens were then strengthened with CFRP sheets in the joint region.

**3.3. Specimen preparation**

In order to make sure that all three specimens were cast identical to one another and that they had the same dimensions and same reinforcements, each specimen was prepared through the same sequence of steps as the other two.

**3.3.1. Preparing the Formwork**

The mould was generally made of timber and was put together by the use of steel connections and screws. The base of the mould was also made of timber and all other components were attached firmly to the base by the use of steel angles. Figure 3-2 shows a preview of the mold and the connections used to hold it together.

![Figure 3-2 Timber mold and the steel connections used to hold it in place](image-url)
Once all the components of the mould were put together, the openings between connections and also the gap between the base and other members were sealed to prevent the concrete from losing any water due to leakage. Another point taken into consideration while preparing the mould was the point where cyclic loading was supposed to be applied at. A small hole was cut out in the mould on the top side of the beam where a steel plate was attached later on. Moreover, to avoid having sharp edges in the beam cross section in the vicinity of the beam-column connection to facilitate the FRP strengthening process, two concave circular foam segments were placed inside the mold and glued to the surface as shown in Figure 3-3.

![Figure 3-3 Foam segments placed inside the mold in the vicinity of the joint](image)

### 3.3.2. Steel Reinforcements

The nominal properties of steel reinforcements intended for this study are shown in Table 3-1. Nonetheless, the actual tensile strength of both longitudinal rebars and stirrups were tested using the Instron-4300 in the Highbay laboratory at the University of Wollongong.

<table>
<thead>
<tr>
<th>Type of Reinforcement</th>
<th>Designation</th>
<th>Nominal Tensile Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal (Beam)</td>
<td>N12</td>
<td>500</td>
</tr>
<tr>
<td>Longitudinal (Column)</td>
<td>N16</td>
<td>500</td>
</tr>
<tr>
<td>Shear</td>
<td>R10</td>
<td>250</td>
</tr>
</tbody>
</table>
Once the mould was ready, steel reinforcements needed to be assembled and tied firmly. However, first the stirrups and longitudinal bars which were designed to have strain gauges installed on them were to be prepared. For that purpose, surface preparation was performed on selected bars and stirrups by the use of electric grinders and sand paper as shown in Figure 3-4. Other reinforcements were also checked for unclean surfaces and were cleaned if necessary. Stirrups, in particular, were all cleaned by the use of sand paper and were then put in place and fixed at intended spaces shown above in Figure 3-1.

![Figure 3-4 Surface preparation for the installation of strain gauges on steel reinforcements](image)

The next step before placing the assembled steel reinforcements inside the mold was installing mild steel strain gauges onto stirrups and longitudinal rebars. Strain gauges were glued to the prepared surfaces by the use of Cyanoacrylate adhesive as shown in Figure 3-5.

![Figure 3-5 Mild steel strain gauges installed on stirrups and longitudinal reinforcements](image)

A total of eight mild steel strain gauges (FLA 5-11-3LT) were attached onto reinforcing longitudinal bars and stirrups in the joint region of all three specimens. A schematic image of the installed strain gauges on steel reinforcements is presented in Figure 3-6.
Figure 3-6 Strain gauge lay out in the joint area of specimens

At the final stage of steel reinforcement preparation, the assembled rebars were placed inside the mould and separators were put underneath to ensure the minimum concrete cover required by AS-3600-2009. In addition, the resistance of all strain gauges was tested by a multi-meter and the wires connected to strain gauges were secured so that they would not be damaged while pouring the concrete. At last, the loading plate was also attached and welded to the top side of the beam.

3.3.3. Casting of the specimens

The strength of the concrete used to cast all three specimens was specified at 32 MPa; however, to determine the actual strength of each batch of the concrete, 9 standard cylinders were cast to be tested for 7-day and 28-day strengths and also for concrete strength on the day of testing the T-sections.

Before casting each specimen, the slump test was also performed according to AS1012.3.1-1998 to ensure the slump amount was within the allowable limit provided by AS-3600-2009. In order to carry out the slump test, the cone was filled with concrete in three layers, each layer being
tamped 25 times. Once the cone was removed, the height of the slump was measured and recorded. The results of the slump tests for each batch of the concrete are presented in Table 3-2.

<table>
<thead>
<tr>
<th>Concrete Batch No.</th>
<th>Specimen</th>
<th>Slump (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Batch No.1</td>
<td>SP1 (Control Specimen)</td>
<td>140</td>
</tr>
<tr>
<td>Batch No.2</td>
<td>SP2</td>
<td>150</td>
</tr>
<tr>
<td>Batch No.3</td>
<td>SP3</td>
<td>140</td>
</tr>
</tbody>
</table>

Once slump measurements were found to be satisfactory, the pouring of the concrete started. Throughout the process the concrete was compacted with the use of mechanical vibrators and tamping bars in order to ensure there would be no voids in the concrete and also to remove any trapped air. Then, the surface of the concrete was troweled and excess concrete was removed from the sides of the mould.

The curing of the specimens was conducted by the use of wet burlaps. The specimens were covered as shown in Figure 3-7 and the water content of the burlaps was replenished every day in order to keep the concrete moist and also to prevent temperature changes which could result in evaporation of the water content of the specimen. Due to the geometry of the specimens, they were kept in the mould for a period of seven days after which they were removed from the mould and placed in a cool and covered part of the laboratory where the curing phase continued. The concrete cylinders on the other hand were removed from the molds after 24 hours and placed in a curing tank in the Highbay lab for the curing phase.
3.3.4. Cross-section Modifications

As discussed in Chapter 2, the results of the tests reported by Mirmiran et al. (1998) and Ghobarah & Said (2002) indicated that CFRP reinforcements which were applied to rectangular cross sections with sharp edges could not provide enough confinement to the joint area and that stress concentration in sharp corners of the cross sections would prevent FRP composites from reaching their ultimate capacity. Consequently, measures were taken to alter the cross sections of specimens SP2 and SP3 in the vicinity of the beam-column connection from rectangular to circular sections. For this purpose, circular segments were cast separately in order to be attached to the specimens. Prior to attaching the circular segments, surface preparation needed to be carried out on the concrete in the joint region. By using electrical grinders and sand paper, excess concrete was removed from the surface and the joint region was given a smooth finish to ensure that the circular segments would be firmly attached to the joint. After surface preparation was performed, circular segments were joined to three sides of the column with a mixture of Epoxy and filleting blend as adhesive. Once put in place, the segments were strapped tightly in order to allow for the adhesives to dry. Epoxy was also used to fill any remaining gaps between the segments and the specimen and if necessary excess adhesives were cleared or grinded off once dried. In order to facilitate the CFRP strengthening process circular segments for the joint face of the column were attached after CFRP wraps were applied to the joint area. Figure 3-8 exhibits the modified cross section of specimen SP2.
3.3.5. CFRP Strengthening
Out of the three specimens designed for this study, SP1 was the control specimen with no CFRP strengthening, SP2 and SP3 were reinforced with CFRP sheets. The strengthening schemes for these two specimens are shown in Figure 3-9 and Figure 3-10.
The characteristics of the carbon fibers used in this study are shown in Table 3-3. The CFRP strengthening procedure was carried out according to ACI 440.2R-2008 standards. After cleaning the surface of the concrete for the application of CFRP wraps, epoxy was applied to the surface in a way to cover the whole area where CFRP wraps were to be attached and once CFRP wraps were put in place epoxy was applied onto the composites to cover the whole strip. Afterwards, an aluminum roller was used to remove any air bubbles which might have been trapped underneath the surface of the CFRP wrap without removing any of the epoxy. Then, in the case of specimen SP3 the next layer of composite was applied and this process continued until all necessary layers were attached to the specimens.

### Table 3-3 Carbon FRP Labelled characteristics

<table>
<thead>
<tr>
<th>Properties</th>
<th>Nominal Amount in Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness</td>
<td>0.34 mm</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>3.79 GPa</td>
</tr>
<tr>
<td>Tensile Modulus</td>
<td>230 GPa</td>
</tr>
<tr>
<td>Ultimate Elongation</td>
<td>1.7 %</td>
</tr>
<tr>
<td>Density</td>
<td>1.73 g/cm³</td>
</tr>
</tbody>
</table>
Throughout the strengthening process, all OH&S and MSDS regulations on fiber reinforced polymers and epoxy products were taken into consideration and special masks and gloves were provided by technicians in the SMART lab at the University of Wollongong. Figure 3-11 demonstrates the progress of the application of CFRP sheets to SP2.

As mentioned above, circular concrete sections were not applied to the joint face of the column before in order to leave enough room for proper CFRP attachment to the beam face of the connection. Therefore, once beam face of the joint was wrapped and allowed time to dry, the remaining of the concrete segments were attached by the use of structural adhesives and were then strapped to ensure precise positioning as shown in Figure 3-12.
The column sections on either side of the beam were also reinforced with CFRP sheets after the concrete segments modifying the cross section of the column to a circular section had been fixed in place. Nevertheless, prior to CFRP application these surfaces also needed to be grinded and cleaned and any excess adhesive on the surfaces was removed. Figure 3-13 shows the final result of CFRP strengthening of Specimen SP2.

![Specimen SP2 reinforced with external CFRP sheets](image)

The same sequence of actions was used in the external reinforcement of Specimen SP3. Since Specimen SP3 was strengthened with six times as many layers of CFRP sheets as Specimen SP2, more preparation was needed and as air bubbles were more likely to remain between layers of composite sheets, aluminum rollers had to be used more frequently to ensure all bubbles were removed from the specimens to create a smooth finish.

### 3.4. Experimental set up

A testing frame in the SMART lab at University of Wollongong was used to carry out the tests on the three specimens. The specimens were placed in the frame with the help of a crane and were then fixed in the frame by the use of bolts and cables. Figure 3-14 presents the testing scheme used in this study. The actuator applied a push and pull loading cycle onto the beam at a distance of 1100 mm from the joint.
According to the research design, a displacement controlled cyclic load would enforce ± 10 mm deflection at the loading point in the first cycle at a rate of 5 mm/ min. After each cycle, the specimen would be checked for cracks and general health. In each cycle, the target displacement enforced by the actuator would increase by 10 mm. This process would continue until the specimen would fail and a sudden decrease in the applied load would be noticed. Upon failure, the loading phase would come to an end and the specimen would be checked yet once more for visible cracks, spalling of concrete, ductile and brittle behavior of the joint, steel reinforcement deformation if visible, CFRP de-bonding (in case of Specimens SP2 and SP3), CFRP breakage and also any kind of damage to the composites which would indicated concentrated tension. Figure 3-15 presents a cyclic load history of the tests at the tip of the beam on the three specimens.
Data collection was done with help of strain gauges, inclinometers and LVDTs (Linear Variable Displacement Transducer). As explained before, two sets of strain gauges were used; mild steel gauges and CFRP gauges. Two LVDTs were used to record displacements on the beam; one monitored the deflection of the beam at 300 mm away from the joint region in the beam area and the other one was placed inside the actuator and measured displacement at the location of the applied load. Three inclinometers were used on the joint area for the control Specimen, SP1, and one inclinometer was used on the side of the beam in the vicinity of the joint for Specimens SP2 and SP3. Figure 3-16 shows the attached inclinometers for SP1.
All strain gauges, inclinometers and LVDTs were hooked into a data collector hub after being calibrated. The actuator also needed to be calibrated for the specific loading scheme. Nonetheless, the day before the testing date of the control specimen and during the calibration of the actuator, a predicament emerged which caused the actuator to overload and break the specimen as shown in Figure 3-17.

![Figure 3-17 Control specimen damaged by the actuator](image)

This was a huge set back in the progress of the experiment. All the materials needed to cast a new specimen were ordered and another control specimen with all the same characteristics, geometry, reinforcements and strain gauges, was cast and cured.

After taking all the necessary precautions to prevent another dilemma and once all the testing equipment was calibrated by the technicians in the lab, the new control specimen was placed in the testing frame and fixed in place as shown in Figure 3-18. The vicinity of the joint region was brushed with a coating of white paint in order to make it easier to observe and mark cracks as they generate and propagate throughout the loading phase.
Afterwards, all data collectors were checked for accuracy again and the loading phase began. Because of the fact that the control Specimen, SP1, had no CFRP sheets around the joint region, the observation of crack formations and crack propagation and also failure modes during the test was possible. Consequently, after each displacement cycle, cracks were marked and the overall behavior of the specimen was monitored. In order to be able to distinguish between the cracks formed in the push phase of the loading cycle and the cracks formed in the pull phase of the loading cycle, two different colors were used to mark the cracks. As shown in Figure 3-19 cracks marked in blue are the cracks formed in the push cycle and the red marker indicates cracks formed in the pull cycle. As displacements got larger and larger, cracks started to propagate from either side of the joint to the back side of the column where they grew wider and wider spalling of the concrete cover in the column began. As can be seen in Figure 3-19 the cracks formed in the push cycle seem to have spread further and wider than those generated in the pull cycle of the loading phase.

Figure 3-18 Specimen SP1 securely placed inside the testing frame
As can be seen in Figure 3-19, the shortage of shear reinforcements in the joint region caused cracks to form rather quickly which later on during the test led to a semi brittle failure of the joint. The brittle failure of the specimen in the joint region was followed by complete spalling of the concrete cover in the column face of the joint.

After the specimen was detached from the actuator, the debris was removed and Specimen SP2 was placed in the testing frame. Since the joint area of Specimen SP2 was covered by CFRP sheets only one inclinometer was used on the side of the beam to measure the curvature of the beam in the vicinity of the joint. Similar to preparations before testing SP1, measures were taken for the calibration of data collectors and other equipment. Although the overall behavior of the joint in SP2 was not as conspicuous as the control specimen due to the presence of CFRP sheets, the specimen was still checked after each loading cycle for sign of failure and spreading cracks. Figure 3-20 shows a picture of Specimen SP2 during the test.
This specimen also was tested to failure. Contrary to the control specimen, the failure mode of this specimen was not a brittle one but a more ductile failure. The beam-column connection started making light cracking sounds in the third cycle. The propagation of cracks continued until the CFRP reinforcements finally gave in and the specimen reached failure as shown in Figure 3-21.

As can be seen in Figure 3-21, the CFRP reinforcement prevented any possible spalling of the concrete and also rendered a ductile failure rather than a brittle one. The same routine was enacted once more for the third Specimen SP3 and after all calibrations the testing of SP3 began. In the first 5 cycles, no sign of damage was seen or heard from the specimen, it was in the 6th cycle, though, that the first cracking sounds started to be heard. However, before these cracks
could cause any damage to the beam-column connection, the beam failed at the boundary of CFRP sheets near the joint while the joint seemed undamaged.

3.5. Summary

This study focused on the efficacy of applying single and multiple layers of CFRP sheets to reinforced concrete beam-column connections as a method of strengthening against shear failure under seismic loadings. Three reinforced concrete T-connections were designed and cast with no transverse shear reinforcements in the joint region. One specimen was used as the reference or control specimen, SP1. SP2 and SP3 were reinforced with external CFRP sheets. Nonetheless, before the application of CFRP composites, SP2 and SP3 had their joint cross sections altered from rectangular to circular sections in order to avoid any concentrated tensions. SP3 was reinforced with six times as many layers of CFRP as SP2. The reason behind that was to identify whether the number of layers of fiber reinforced polymer sheets would have any effect on the overall behavior of the specimen or the ultimate strength of the joint. The primary results of the tests and also the observations showed that the control specimen, with no shear reinforcement in the joint and no external CFRP reinforcement, failed rather quickly and exhibited brittle failure. SP2 with only 1 layer of CFRP showed a higher ultimate strength and underwent a ductile failure mode. SP3, with six layers of CFRP reinforcement, had the highest ultimate strength and failure occurred in the beam away from the joint. The quantitative analyses of the results are presented in Chapter 4: Results.
Chapter 4 Results

4.1. Introduction

This chapter presents all the results from the experiments carried out on the specimens in this study. Firstly, the results of the control tests on the three specimens are presented and compared and then the results of cyclic loading tests on the beam-column connections are demonstrated in the form of graphs and tables followed by strain gauge and LVDT readings for the specimens.

4.2. Control Tests

Despite the fact that all three T-connection specimens were initially cast identical to one another in terms of steel reinforcements, nominal strength of the concrete and dimensions of the specimens, standard concrete cylinders were made on the day of casting of each specimen in order to compare the actual concrete strength of the samples in order to ensure a fair analysis of results. For each of the control tests, three concrete cylinders were used and the average of the three compressive strengths and also the average cylinder diameter were used as reference. Compressive strength tests were performed on the 7th day and the 28th day after casting and also on the testing day of the specimens according to AS 1012-1998. The results of the control test are presented in Table 4-1.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>7th day Compressive Strength (MPa)</th>
<th>Average Cylinder Diameter (mm)</th>
<th>28th day Compressive Strength (MPa)</th>
<th>Average Cylinder Diameter (mm)</th>
<th>Testing day Compressive Strength (MPa)</th>
<th>Average Cylinder Diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP1</td>
<td>22.05</td>
<td>100.50</td>
<td>36.12</td>
<td>100.50</td>
<td>40.91</td>
<td>100</td>
</tr>
<tr>
<td>SP2</td>
<td>22.11</td>
<td>101</td>
<td>33.17</td>
<td>101.25</td>
<td>41.43</td>
<td>100.50</td>
</tr>
<tr>
<td>SP3</td>
<td>28.62</td>
<td>99.50</td>
<td>40.02</td>
<td>100</td>
<td>44.25</td>
<td>99.80</td>
</tr>
</tbody>
</table>

The three specimens used in the study were cast at different times and with different batches of concrete. The reason for that was the scale and dimensions of the specimens and the fact that...
casting all three at the same time would require three times the space in the laboratory. Therefore, once the curing phase of one specimen was over, the casting of the next specimen would begin.

The actual tensile strengths of steel reinforcements used in the study were also measured with the help of the Instrone-4300 machine in the Highbay Lab at the University of Wollongong. For each of the longitudinal reinforcements and stirrups, three samples were tested. The stress-strain diagrams for the longitudinal reinforcements and also the stirrups are presented in Figures 4-1, 4-2 and 4-3.

![Figure 4-1 Stress-Strain Curve for stirrup reinforcement R10](image-url)
4.3. Cyclic Loading Tests

During the cyclic loading test of each of the three beam-column connections, different characteristics and behaviors of the Specimens were monitored and recorded; the following sections represent the data collected on the specific behavior of members in each specimen individually and also in comparison with their counterparts in the other two beam-column connections.
4.3.1. Ultimate Strength

National Instruments Model PXIe – 1078 data collecting device was used throughout the tests to record the applied load to the specimen at all times. The cyclic loading history for the control Specimen, SP1, which had no CFRP reinforcement and also no transverse steel reinforcement in the joint region, is illustrated in Figure 4-4. The vertical and horizontal axes in the chart represent the applied load to the specimen and time, respectively. For specimen SP1, the maximum applied load in the “Push” phase of the experiment was 35.36 kN and the maximum in the “Pull” phase of the test was 36.23 kN.

As can be seen from the chart, the specimen reached its ultimate capacity in the early stages of the test in the third loading cycle at T=30.1 minutes and the ultimate strength of the specimen declined significantly afterwards.

The results of the same test for Specimen SP2 and SP3 which were reinforced with CFRP sheets are presented in Figure 4-5 and Figure 4-6 respectively.
The loading history graph for Specimen SP2 indicates that the specimen reached its ultimate capacity of 67.57 kN in the fourth loading cycle at $T=73.96$ minutes in the push cycle. The maximum applied load in the pull cycle for Specimen SP2 was 63.32 kN.

For Specimen SP3 which was externally reinforced with six times more CFRP layers than specimen SP2, the maximum applied load was of a magnitude of 88.114 kN at $T=58.78$ minutes in the fourth push cycle while the maximum applied load in the pull cycle was 83.95 kN.
In terms of ultimate strength, Specimen SP2 which was strengthened with CFRP sheets exhibited a capacity 74% higher than that of the control specimen and Specimen SP3, strengthened with 6 times the number of CFRP layers used for Specimen SP2, showed a 143% increase in load carrying capacity compared to the control Specimen SP1.

### 4.3.2. Maximum Deflection at Loading Point

As discussed in Section 3.4. Experimental Setup, two LVDTs were used to monitor displacements at the point of the application of the load and also at a point 300 mm from the joint region on the beam. The LVDT placed inside the actuator, recorded the imposed deflection on all the specimens.

Since the same displacement control loading history was used for all three specimens as shown in Figure 3-15, the graphs for maximum displacement at the loading point for all three specimens are close to each other as shown in Figures 4-7, 4-8 and 4-9.

Figure 4-7 demonstrates the displacement applied to control Specimen SP1 with the peak displacement at 96.45 mm 245.3 minutes after the beginning of the test.

![Figure 4-7 Displacement history for control Specimen SP1](image)

Figure 4-8 indicates that the maximum imposed displacement to specimen SP2 was 90.68 mm at 354.5 minutes into the test.
The graph for Specimen SP3 shows that the specimen underwent a maximum displacement of 90.20 mm at 282.2 minutes after loading had started as shown in Figure 4-9.

The main reason for recording and also presenting the results of the actuator LVDT readings were to ensure that all specimens had undergone the same loading pattern as intended and that the comparisons and analyses that were made were fair ones.

Load-displacement comparisons also provide a good understanding of the behavior of the specimens throughout the test. Figures 4-10, 4-11 and 4-12 demonstrate the applied load vs. beam tip displacement for Specimens SP1, SP2 and SP3 respectively.
Figure 4-10 Applied load vs. beam tip displacement for control Specimen SP1

Figure 4-11 Applied load vs. beam tip displacement for Specimen SP2
4.3.3. Displacement in Joint Vicinity

In order to monitor the behavior of the joint during the test, LVDTs were installed to not only measure the displacement at the tip of the beam but also to record deflections at several points in the vicinity of the joint. The points chosen for this matter were a point on the beam 300 mm from the face of the joint and two points on the column 300 mm from the joint at either sides of the joint as shown in Figure 4-13.
Point B Vertical Displacement

Vertical displacement at point B which was placed 300 mm away from the joint on the beam was recorded for the three specimens. For the control specimen only the maximum displacement at point B was recorded but for specimens SP2 and SP3 a detailed reading was performed through the test which are presented in Figures 4-14 and 4-15.

![Figure 4-14](image1)

**Figure 4-14** Vertical displacement history at point B (refer to Figure 4-13) Specimen SP2

![Figure 4-15](image2)

**Figure 4-15** Vertical displacement history at Point B (refer to Figure 4-13) Specimen SP3

For specimen SP1 which was the control specimen, the maximum displacement at point B was 24.8 mm. The maximum displacement at Point B for Specimens SP2 and SP3 were recorded at 25.15 mm and 30.66 mm, respectively.
Points A and A´ Horizontal Displacements

The horizontal displacements at Points A and A´ on the column faces of the Specimens SP1, SP2 and SP3 were recorded and the peak values at the time of the application of ultimate load to the specimen are presented in Table 4-2.

Table 4-2 Peak displacements at Points A and A'(refer to Figure 4-13)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum Displacement at Point A</th>
<th>Maximum Displacement at Point A´</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control specimen SP1</td>
<td>9.28 mm</td>
<td>8.3 mm</td>
</tr>
<tr>
<td>SP2</td>
<td>7.6 mm</td>
<td>7.05 mm</td>
</tr>
<tr>
<td>SP3</td>
<td>6.58 mm</td>
<td>5.9 mm</td>
</tr>
</tbody>
</table>

4.3.4. Rotation at Joint

The data recorded from the inclinometers installed on the face of the joint for each specimen indicated that the specimens which were strengthened with CFRP sheets, SP2 and SP3, underwent a larger rotation at the joint compared to the control specimen. Figure 4-16 shows the data readings for rotation at joint for the control Specimen SP1.

![Figure 4-16 Rotation at joint history for control Specimen SP1](image-url)
The control Specimen SP1 underwent a maximum rotation of 5.38 degrees at 245.3 minutes through the test.

As shown in Figure 4-17, for Specimen SP2 the largest value of the imposed rotation at joint was -8.22 degrees at 219.5 minutes after the initiation of loading.

For Specimen SP3, the inclinometer recorded a maximum rotation of 7.90 degrees which was marked at 338.60 minutes into the test. As can be seen in Figure 4-18, in the joint rotation history graph for SP3, in the 3rd and 4th cycles two unusual flat areas can be observed. The reason behind
the two anomalies was corrupted data logged by the data logging device (National Instruments Model PXle – 1078 which caused the graph to flatline in those two instances.

4.3.5. Strain Gauges

The strain gauges which were installed on to the steel reinforcements for all specimens and also the strain gauges which were mounted onto the CFRP reinforcement for specimens SP2 and SP3 were connected to two data collector hubs which recorded strains every second throughout the test. The positions of the aforementioned strain gauges are depicted in Figure3-6 for steel strain gauges. Table 4-3 contains the steel reinforcement strain readings at the time of yielding for each of the specimens. The highlighted data present strain readings higher than the yield strain of the steel reinforcement. The yield strains were calculated for the longitudinal steel reinforcements and stirrups by using the stress-strain curves acquired through the tensile test of the steel reinforcements as shown in Figures 4-1, 4-2 and 4-3. The yield point on the stress-strain diagram of the steel rebars was determined by using the 0.2 % offset on the diagram with the slope of the linear section to estimate yield stress and strain for each steel bar. As a result the yield strain for the stirrup reinforcement R10 was calculated at 1560 micro strain, for the longitudinal rears in the beam it was around 1850 micro strain and for the column longitudinal reinforcement strain at yield was approximately 2140 micro strain.

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Micro Strain at Yield</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Control Specimen SP1</td>
</tr>
<tr>
<td>Strain Gauge 1</td>
<td>4.54</td>
</tr>
<tr>
<td>Strain Gauge 2</td>
<td>13.1</td>
</tr>
<tr>
<td>Strain Gauge 3</td>
<td>1429</td>
</tr>
<tr>
<td>Strain Gauge 4</td>
<td>1088</td>
</tr>
<tr>
<td>Strain Gauge 5</td>
<td>823</td>
</tr>
<tr>
<td>Strain Gauge 6</td>
<td>2166</td>
</tr>
<tr>
<td>Strain Gauge 7 (Stirrup)</td>
<td>2398</td>
</tr>
<tr>
<td>Strain Gauge 8 (Stirrup)</td>
<td>2076</td>
</tr>
</tbody>
</table>
The locations of all strain gauges were demonstrated in Figure 3-6. Table 4-4 presents the ultimate strain gauge readings for steel strain gauges in each specimen.

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Ultimate Micro Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Control Specimen SP1</td>
</tr>
<tr>
<td>Strain Gauge 1</td>
<td>14.8</td>
</tr>
<tr>
<td>Strain Gauge 2</td>
<td>14.6</td>
</tr>
<tr>
<td>Strain Gauge 3</td>
<td>2330</td>
</tr>
<tr>
<td>Strain Gauge 4</td>
<td>2320</td>
</tr>
<tr>
<td>Strain Gauge 5</td>
<td>1120</td>
</tr>
<tr>
<td>Strain Gauge 6</td>
<td>2320</td>
</tr>
<tr>
<td>Strain Gauge 7 (Stirrup)</td>
<td>2330</td>
</tr>
<tr>
<td>Strain Gauge 8 (Stirrup)</td>
<td>2310</td>
</tr>
</tbody>
</table>

4.3.6. Summary

From a quick glance at the results of the displacement control cyclic loading tests, it could be concluded that the specimens which were externally reinforced with carbon fiber reinforced polymer sheets in the joint region, SP2 and SP3, performed more effectively than the control Specimen SP1. The initial results not only indicated that the presence of CFRP sheets around the joint region triggered higher load carrying capacities in the two specimens but also suggested that the number of layers of carbon FRP composites used to retrofit the beam-column connection was also of utmost importance in determining the capacity of the joint. Failure modes and displacements of the specimens were also affected by the strengthening technique used in this study. Specimens with CFRP reinforcements exhibited ductile failure modes rather than the brittle failure which was observed in the case of the control specimen. Rotation and displacement in the vicinity of the joint seemed to have increased as a result of joint rehabilitation by CFRP sheets.

A full analysis of the results and also a more detailed look at the outcome of the experiments is carried out in Chapter 5: Discussion.
Chapter 5 Discussion

5.1. Introduction

In this section, failure modes, load carrying capacities, beam tip displacements, joint rotations and the general behavior of the specimens are analysed. Moreover, the results and charts for each specimen are scrutinised and compared with the other two specimens as well as the studied literature to determine the effects of the presence of CFRP sheets around the joint region and also to determine whether the modification of the joint cross section influenced the performance of specimens SP2 and SP3. In the end, an overall conclusion is made and recommendations for future work are presented.

5.2 Failure Modes

Once each of the specimens was placed inside the testing frame, their overall behavior was monitored and crack formation and propagation were observed closely until the specimens reached the point of failure. For the Control Specimen SP1, cracks started to form quite early in the test in the 2nd loading cycle and spread through the joint region. The failure mode of SP1 was a brittle one; which was followed by complete spalling of concrete in the joint region due to absence of shear reinforcements inside the joint as shown in Figures 5-1 and 5-2.
Specimen SP2 was also tested to failure. Contrary to the control specimen, the failure mode of this specimen was not a brittle one but a more ductile failure due to the presence of CFRP reinforcements in the joint region similar to the reported results by Amoury and Ghobarah (2001). The beam-column connection started making light cracking sounds in the third cycle. The propagation of cracks continued until the CFRP reinforcements finally gave in and the specimen reached failure as shown in Figure 5-3.
As can be seen in Figure 5-3, the presence of CFRP sheets around the joint area of Specimen SP2 prohibited any spalling of concrete. However, cracks still formed and spread through the joint starting from the beam face of the joint and propagating towards the column which eventually caused the specimen to give away under loading.

The failure mode for Specimen SP3 was totally different to those of SP1 and SP2. Not only did the presence of CFRP sheets stop the spalling of concrete for Specimen SP3, it also rendered the joint so strong that the failure happened in the beam rather than the joint as can be seen in Figure 5-4.

The results of the observations indicated that the confinement provided to the specimen by the presence of CFRP sheets delayed the formation of cracks and also hindered the propagation of the cracks formed in the vicinity of the joint; which means that CFRP composites compensated for the absence of steel reinforcements in the joint region. In addition, the comparison between the failure modes of Specimen SP2 and Specimen SP3, which had six times as many CFRP sheets around the joint area as SP2, shows that the number of CFRP layers applied or the composite density had direct impact on the performance of the specimen. The overall behavior and failure mode improvement of the specimens with the presence of CFRP sheets are supported
by findings by Gergeley et al. (2000) and Lee et al. (2010) who reported an improved overall shear behavior of the joint as a result of the confinement provided by FRP composite sheets.

### 5.3. Ultimate Strength

The loading history of the three specimens, indicated that Specimen SP3 had the highest ultimate strength among the three, followed by Specimen SP2 and the control Specimen SP1.

![Figure 5-5 Ultimate Ultimate Strength of specimens](image)

The presence of Carbon FRP sheets in Specimen SP2 caused an 86% increase in the ultimate strength of the specimen compared to that of the control specimen. In similar studies by Amoury and Ghobarah (2001) and Niroomandi et al. (2010), the reported results indicate a 52% and 66% increase, respectively, in the shear capacity of the retrofitted specimens.

Specimen SP3, compared to the control Specimen SP1 exhibited a 143% increase in its load carrying capacity. The presence of extra CFRP layers around the joint area also triggered an increment of 30% in the ultimate strength of Specimen SP3 in comparison with Specimen SP2. These findings supported previous reports by Dalalbashi et al. (2012) that the number of CFRP layers used to provide confinement to the joint region plays a significant role in the overall load carrying capacity of beam-column connections.
Another improvement in the performance of the CFRP strengthened specimens can be observed in Figure 5-6.

![Figure 5-6 Specimen Ultimate Strength after yielding](image)

Figure 5-6 demonstrates the ultimate strength of the three specimens once they had reached their ultimate capacity. The points marked on the graph are the peak points of the cyclic loading history graphs shown in Figures 4-4, 4-5 and 4-6. As can be seen in the graph, for the control specimen SP1, once the specimen yields, the ultimate strength of the frame starts to decline almost immediately from the next loading cycle. The load carrying capacity of the control specimen dropped by 13% in the first cycle after yielding and from then on the capacity declined by an average of 15% in each cycle.

In the first cycle after reaching the ultimate capacity, SP2 lost 9% of its strength, in the second cycle, the specimen’s strength reduced by another 18% and from then on, in each cycle the ultimate strength of the joint declined by an average 20%.

Specimen SP3, however, performed differently to the other two specimens. As can be observed in Figure 5-6, in the first three cycles after reaching its ultimate capacity, the ultimate strength of Specimen SP3 was only reduced by 1% in each cycle. In the 4th and 5th cycles, the capacity of the beam-column connection decreased by 4% and 20% respectively.
This was due to the fact that the confinement which was supplied to the joint by the CFRP sheets prevented the connection from losing its original capacity after yield and as a result kept the integrity of the beam-column joint intact.

5.4. Displacements in the Vicinity of the Joints

The readings from the LVDTs placed in the vicinity of the joint at Points A and A’, which were marked at 300 mm at either sides of the joint on the column, and also at Point B, which was located at 300 mm from the joint on the beam surface, were presented in Chapter 4: Results.

The readings indicate that the presence of CFRP sheets in the joint region hindered displacements at Points A and A’. A comparison between the three specimens shows that there was an 18% decrease in the displacement at Point A for SP2 compared to the control specimen and also a 30% reduction in the maximum displacement at the same point for SP3. For Point A’, these values were 15% and 28% reduction in displacement compared to the control specimen for Specimens SP2 and SP3 respectively.

The trend for the vertical displacement at Point B, however, was the exact opposite. SP3 had the highest displacement at B with 30.66 mm followed by SP2 at 25.15 mm and the control Specimen SP1 at 24.80 mm. This is attributed to the higher joint stiffness as a result of CFRP reinforcement which was also reported by Li et al. (1999) and Lee et al. (2010). The increased value for member stiffness caused the joint to retain its integrity as the beam section plus the column face of the joint through the test. As it can also be noted in Figures 4-7, 4-8 and 4-9, Specimen SP3 had the highest displacement at yield, followed by SP2 and the control Specimen SP1. Mukherjee et al. (2004) also reported an increased displacement at yield for specimens retrofitted with carbon FRP compared to control specimens.

Figure 5-7 is a sketch of a deformed beam-column connection without external reinforcement to the joint.
As can be seen in Figure 5-7, the applied load at the beam tip causes a deformation in the joint region, which in turn results in displacements at Points A, A’ and B. When the joint area was reinforced with CFRP sheets, nonetheless, the heightened stiffness at joint resisted against the displacements at Points C and D which in turn reduced the drift at Points A and A’. Vertical displacement at Point B, on the other hand, increased due to the fact that with an increased stiffness at joint, the deformed shape of the beam face of the joint became more linear which in turn when accompanied with the imposed displacement at beam tip, caused a larger displacement at Point B.

The same reasoning can be applied to joint rotation recordings. Similar to findings by Mahmoud et al. (2012), results of this study indicate that with an increased stiffness at joint, for the same value of imposed displacement, larger rotation values were recorded in the joint region.
5.5. Strain Gauge Readings

The strain gauge readings at yield and also the ultimate strains for steel reinforcements in the joint region were presented in Table 4-3 and Table 4-4 respectively. For strain gauges 1 and 2, which were located on the column reinforcing steel at the top corner of the joint, as shown in Figure 5-8, the ultimate strains and the strains at yield were larger in the strengthened specimens than in the control specimen. Moreover, Specimen SP3 which was reinforced with a larger number of CFRP sheets exhibited larger strains in strain gauges 1 and 2 than Specimen SP2. Moreover, the results show that in Specimen SP3 at the moment of specimen yield both strain gauges recorded readings higher than the yield strain of the steel reinforcement which means that the external reinforcement by CFRP sheets triggered an improved performance by the concrete in the joint region which allowed for the steel reinforcement to reach its yield capacity. In Specimen SP2, the recorded strain at the moment of specimen yield for SG1 and SG2 were much higher than those for the Control Specimen SP1, as shown in Figure 5-9, however, the steel did not reach its yield strain. This proves that the presence of multiple layers of carbon FRP composites, resulted in a better performance of the joint under cyclic loading.

Figure 5-8 Location of strain gauges 1 and 2
For strain gauges 3 and 4 which were mounted on beam steel reinforcements in the top corner of the beam-column joint, as shown in Figure 5-10, the result of the strain readings at yield and also the ultimate strains for Specimen SP3 were still much higher than those for the Control Specimen and Specimen SP2. Nonetheless, Specimen SP2 exhibited lower strains in SG3 and SG4 compared to the Control Specimen as can be seen in Figure 5-11.
The beam steel reinforcement at the location of SG3, reached its yield capacity in Specimen SP3 while in the Control Specimen and also SP2 the values for ultimate strains were lower than the yield strain of the steel rebar. This fact, once more, confirms the previous findings on the efficacy of the application of several CFRP layers to the joint region.

Strain gauges 5 and 6 which were also mounted on beam steel reinforcement inside the joint, as shown in Figure 5-12, exhibited similar results to those by SG 3 and SG 4.

Strain gauge readings for SG5 and SG6 also revealed higher strains in SP3 compared to the other two specimens, as illustrated in Figure 5-13. Moreover, strain gauges for Specimen SP3 reported, once again, strains higher than the yield strain of the steel longitudinal reinforcement which was calculated at 2750 (micro strain).
The results of the strain gauges mounted on beam steel reinforcements reflect the adequacy of the strengthening technique by the use of CFRP sheets. The presence of CFRP sheets around the joint area allowed the concrete to reach its limit and as a result produce higher strains in the reinforcing steel at the time of specimen yield. Comparing the results of the two carbon fiber reinforced Specimens, SP2 and SP3, reveals that the presence of multiple layers of CFRP triggered much better performance by the joint and its adjacent components. The strain gauge readings reported that all the strain gauges which were installed on the longitudinal steel reinforcements in the joint region reached their yield strains which, in turn, demonstrate the utilisation of the full capacity of the connection as a result of the presence of external CFRP reinforcements. The findings in this study were similar to findings by Mahmoud et al. (2012) who also reported that in their study, specimens strengthened with CFRP showed better performance by the concrete section of the joint which resulted in higher values of strains in the steel reinforcements in the joint region.

As for strain gauges 7 and 8 which were located on stirrups in the column just above the connection, the control specimen showed highest value at the time of specimen yield while SP3 had the highest ultimate value amongst the three.

After taking a closer look at the strain gauge readings for the beam longitudinal steel reinforcements and making comparisons amongst the three specimens, another interesting conclusion could be made. The results of the readings from SG3, SG4 which were mounted on the beam longitudinal reinforcements just at the face of the joint, illustrated that in the push and pull cycles imposed on the specimens, the presence of the CFRP sheets around the joint region

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Figure 5-13 Ultimate strains and Strains at Yield for SG5 and SG6 (refer to Figure 5-12)
had a direct impact on the steel reinforcement involvement in contributing to the load carrying capacity in the compression stage of the loading. Figures 5-14, 5-15 and 5-16 demonstrate the strain readings for SG3 over time for the Control Specimen, SP2 and SP3.

Figure 5-14 Strain readings for SG3 over time for the Control Specimen SP1

Figure 5-15 Strain readings for SG3 over time for SP2
As can be seen in Figures 5-14, 5-15 and 5-16, SG3 in the control specimen underwent both tension and compression in a cyclic manner as the specimen was put through a push and pull cycle. However, with only one layer of CFRP sheets applied to Specimen SP2, under the same kind of loading, SG3 readings showed that while the steel reinforcement in the beam still carried both tensile and compressive loads, the ratio of the negative area covered by the graph to the positive area covered was much smaller than that for the Control Specimen SP1. This could be attributed to the fact that the presence of the CFRP sheet provided more confinement to the concrete which increased the compressive strength of the concrete triggering a reduction in the steel reinforcements’ share of the load.

This fact can be seen more vividly for SP3 which was reinforced with six times the number of steel reinforcements in SP2. In Specimen SP3, the strain readings did not indicate any compression in the steel reinforcement in the vicinity of the joint; meaning that once in the compression zone, the entire compressive load was carried by the concrete and minimal tensile stress was recorded for that specific rebar.


Chapter 6: Conclusion

Based on the studied characteristics of the beam-column connections and also the considered defect which was the absence of transverse steel reinforcements in the joint region of the frame, the following deductions were made:

- The application of CFRP sheets in order to strengthen the joint region of a beam-column connection showed its efficiency in improving the overall behavior of a beam-column connection under cyclic loading. The CFRP retrofitted specimens exhibited ductile failure modes rather than brittle failure which was observed in the case of the control specimen.

- The ultimate load carrying capacity of the strengthened specimens was significantly enhanced. Specimen SP2 which was reinforced with a single layer of CFRP in the joint region showed an 86% increase in its load carrying capacity. The study also confirmed findings by Dalalbashi et al. (2012) which stated that the number of CFRP layers applied to the joint is of grave consequence in determining the ultimate load carrying capacity. SP3 which had six times more CFRP layers applied to its joint area than SP2 demonstrated an increase in the capacity by 143%. The extra layers of CFRP reinforcement in SP3 also rendered the joint so strong that the failure of the specimen was contained to the beam area which retained the integrity of the joint confirming findings by Vatani-Oskouei (2010).

- The modification of the cross section of the joint area in Specimens SP2 and SP3 from rectangular to circular provided more confinement to the joint region which prevented issues reported by Mirmiran et al. (1998) and Yang et al. (2004) regarding tearing and spalling of the composites due to stress concentration in the corner area.

- The presence of CFRP composites triggered a reduction in the horizontal displacement of the column section of the joint while increasing the displacement capacity of the beam at the time of specimen yield. Moreover, external reinforcement of the specimens with CFRP composites increased rotation capacity at the face of the joint.
Steel reinforcement strain gauge readings indicated higher strains at yield for specimens SP2 and SP3 which was a result of the confinement provided to the concrete section of the joint that allowed the concrete to reach its full capacity.
References


