FLEXURAL PERFORMANCE AND MOMENT CONNECTIONS OF CONCRETE-FILLED GFRP TUBES (CFFTS) AND CFFT-ENCASED STEEL I-SECTIONS

by

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Abstract

The first part of this thesis addresses a new hybrid system, concrete-filled FRP tube (CFFT)-encased steel I-sections. The embedded steel section enhances flexural strength, stiffness and ductility, and facilitates connection of the CFFT member to footings or other members. Phase I addresses the flexural behaviour of the system through the testing of beam specimens with GFRP tubes which vary in thickness and laminate structure. The steel section enhances performance considerably, especially ductility, in tubes with cross-ply laminates, where significant sustained reserve strength remains upon fracture of the tube. CFFTs with angle-ply tubes show considerable inherent ductility on their own, although adding the steel section enhances strength and stiffness. Phase II addresses the development of a moment connection through cantilever tests. The connection consists of steel base plates welded to the steel sections, which are embedded into CFFT members at various length-to-span (L_s/L) ratios between 0.1 and 1.0. Three distinct failure modes are observed. At (L_s/L) ratios below 0.17, premature bond failure occurs. At ratios of 0.17 to 0.47, flexural tension failure of the tube occurs just beyond the free end of the steel section. Beyond a 0.47 ratio, the plastic hinge capacity is developed at the fixed end. A simple design-oriented model to predict strengths of the connection at the full range of (L_s/L) ratios is developed and validated. Also, a readily available computer program is adopted to model flexural behaviour of the CFFT-steel member itself.

The second part of the thesis investigates unreinforced CFFT members, with emphasis on moment connections to concrete footings. The study explores the effect of maximum shear and maximum moment, both occurring at the same location, on the ultimate strength of CFFTs. Testing involves simple beams and cantilever specimens with varying shear spans and fixed end arrangements. End conditions consist of either direct embedment into concrete blocks with steel dowels, or mechanical clamping. For the cross-ply GFRP tubes used, the presence of shear at the location of maximum moment near the connection of the cantilevers does not reduce flexural
capacity. Slip can prevent the CFFT member from attaining the potential moment capacity in spite of the tube failing due to tensile rupture.
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\( A_s \) – Cross-sectional area of steel section

\( b \) – Width of the steel section

\( C \) – Compressive force, subscripts can denote if the force is due to concrete, steel, or FRP

\( c \) – Neutral axis depth, as measured from the compression face

\( D \) – Outer diameter of the FRP tube

\( f'_c \) – Compressive strength of the concrete

\( f_{\text{cube}} \) – Compressive strength of a cube of concrete

\( f_r \) – Rupture strength of the FRP

\( f_u \) – Ultimate strength of steel

\( f_y \) – Yield Strength of the steel

\( h \) – Depth of the steel section

\( L \) – Total length of the tested specimen, measured between the centres of the supports for simply supported beams or between the face of the connection and the loading point for cantilevers

\( L_s \) – Length of the steel beam inside the CFFT

\( M \) – Applied moment

\( M_{\text{max}} \) – Maximum moment applied to the specimen at any point in the test

\( P \) – Applied load

\( P_u \) – Ultimate load applied to the specimen

\( R \) – Radius of the tube

\( t \) – Thickness of the FRP tube

\( T \) – Tensile force, can be followed by a subscript describing if the force is located in the steel or FRP

\( t_f \) – Thickness of the flange of the steel section

\( t_w \) – Thickness of the web of the steel section
LIST OF SYMBOLS

\( y \) – Distance from the neutral axis to the centroid of a force

\( \tau_u \) – Bond strength between concrete and steel reinforcing bars

\( \omega \) – Reinforcement index
Chapter 1

Introduction

1.1 General

Fibre reinforced polymers (FRPs) are being used more frequently in many fields, including structural engineering. Concrete-filled FRP tubes (CFFTs) are potentially effective structural members for bridge piers, marine piles, or mono-poles. The hollow, off-the-shelf tubes are used in this case as stay-in-place formwork, in addition to providing longitudinal and circumferential reinforcement. The tube has several advantages over steel reinforcement as it can provide the flexural, shear, and confinement reinforcement simultaneously, without the labour costs and time involved in building conventional rebar cages. The tubes can also be customized in terms of thickness or fibre orientation for individual projects to provide different degrees of longitudinal or hoop reinforcement. Furthermore, FRPs have a higher strength-to-weight ratio than steel and are non-corrosive. Filling the FRP tube with concrete provides compressive resistance and prevents local buckling of the thin tube. The CFFT system may also include internal longitudinal steel reinforcement, as the tube will protect the steel from water infiltration and corrosion, while the steel reinforcement provides ductility and additional strength and stiffness.

Numerous research studies have already been performed to examine the behaviour of CFFTs in flexure with analytical models available to predict the behaviour. Several studies have been undertaken to determine the change in behaviour due to the addition of deformed steel bar reinforcement or prestressing (Fam et al., 2007). In addition to the gains in strength, stiffness, and ductility, adding steel was also shown to minimize slip of the tube relative to the concrete core,
which can be a major factor in the loss of composite action (Cole, 2005). Double-skinned tubes with concrete filling between the two tubes have also been studied, with the inner tube being another FRP tube (Fam, 2000) or a steel tube (Teng, 2004). However, little research has been focused on using stronger structural steel sections, such as W- or S-sections, to reinforce the CFFTs. These sections will not be prone to the local buckling that occurs to the inner tube in the double-skinned tubes, as the W-sections will be fully restrained by the concrete. The axial behaviour of circular CFFT-encased steel W-sections was studied by Karimi (2010); however, no work has been done on the system in flexure. This thesis will examine the flexural behaviour of a hybrid circular CFFT-steel system and compare it to an identical but unreinforced CFFT system.

Potential connections between the CFFT member and the rest of the structure or footing is an important area of research that needs to be further explored for the acceptance of this system as a viable alternative to traditional reinforced concrete. This study examines a new potential moment connection for the CFFT-encased steel S-section by welding the steel section to a steel base plate. This plate could then be bolted or welded to another steel member or cast in a reinforced concrete footing. The effectiveness of the connection is tested along with a study of the required embedment length of the steel W-section in the CFFT member, to achieve the full strength of the CFFT member itself.

Tests have already been performed on embedment as a method of connection between a reinforced concrete footing and plain CFFTs (Lai, 2010). When comparing the capacity of that connection to an idealized, mechanically clamped, moment connection that was tested with CFFTs using the same glass FRP (GFRP) tubes (Mitchell, 2008) it was evident that the connection capacity was lower than the member capacity. This study also addresses both of these connections to determine the cause of the decrease in capacity.
1.2 Objectives

The primary objective of this thesis is to examine the flexural behaviour of CFFT-encased steel S-sections and to compare them to unreinforced conventional CFFTs. The thesis also examines the behaviour of several connection methods for CFFTs and CFFT-encased steel S- or W-sections. Specific objectives include:

1. To determine the enhancement in strength, stiffness, and ductility as well as the change in failure modes in CFFT-encased steel S-sections tested in four-point bending relative to unreinforced CFFTs.
2. To determine the effect of parameters such as FRP tube thickness, laminate structure, and amount of reinforcement on the behaviour of CFFT-encased steel S-sections.
3. To investigate a potential moment connection for CFFT-encased steel S-sections through welding the S-section to a steel base plate, in terms of behaviour and effectiveness at achieving the full capacity of the CFFT member.
4. To observe the various failure mechanisms that occur when varying the embedment length of the structural steel section into the CFFT member, and hence establish the minimum length required to achieve the maximum capacity.
5. To determine the reasons for the decrease in capacity seen in other potential moment connection methods for unreinforced CFFTs as previously tested by Lai (2010) and Mitchell (2008) by looking into the effects of moment and shear as well as the influence of slip at the connection location.

The experimental program occurred in three phases. Phase I involved testing both unreinforced CFFTs and CFFT-encased steel S-sections in four-point bending to compare the
behaviour and examine the test parameters. Phase II tested the CFFT-encased steel sections as cantilevers, with the encased steel section being welded to a plate, and varied the length that the steel section extended into the CFFTs. Phase III examined the connection of unreinforced CFFTs through direct embedment of the CFFT member into a reinforced concrete footing or through mechanical clamping, to determine the reason for a decrease in strength of the connection observed and reported in literature.

1.3 Scope

This thesis reports on flexural beam testing of four unreinforced CFFT and four CFFT-encased steel S-sections. FRP tubes varied in diameter, thickness, and laminate structure. Five cantilever specimens with varying lengths of steel were also tested to examine the moment connection of the CFFT-encased steel S-sections. For the unreinforced CFFT members, three cantilevers and two simply supported beams were tested to assess the moment connection methods, including both bond-slip and flexural analyses.

This investigation also includes two analytical models. One model, for the beams tested in four-point bending, used the computer program Response 2000 (Bentz, 2000), to determine the full flexural response of CFFT-encased steel sections. This analysis includes cracking and non-linearity of concrete, bilinearity of FRPs and plasticity of steel. The second model is developed for the moment connection of the CFFT-encased steel section, to establish the strength and failure mode for various embedment lengths of the steel section into the CFFT member.

1.4 Thesis Outline

Chapter 2: reviews the literature available on CFFTs, both reinforced and unreinforced. General behaviour and previous connection methods are summarized.
Chapter 3: describes tests on structural steel-reinforced CFFTs. Tests include comparisons between reinforced and unreinforced CFFT specimens to determine the general flexural behaviour and the effects of tube thickness, reinforcement ratio, and laminate structure. This chapter also provides details on connection tests, consisting of welding the end of the structural steel section to a base plate, as well as a study on the optimal length of the structural steel inside the CFFT member to achieve the full capacity of the connection. Two analytical models are also described in this chapter.

Chapter 4: presents the moment-resisting connection tests of unreinforced CFFTs, by direct embedment into a reinforced concrete footing and by mechanical clamping of the CFFT. Control tests on CFFT beams tested in three- and four-point bending are also presented.

Chapter 5: provides the conclusions of the studies and presents further areas of study required.

References

Appendix A: provides detailed calculations related to the one of the analytical models presented in Chapter 3.
Chapter 2

Literature Review

2.1 Introduction

This section introduces some of the background and applications for FRPs and CFFT's in structural engineering. Flexural behaviour of unreinforced CFFT's and CFFT's reinforced with steel or FRP bars is described. Prestressing of CFFT's is also discussed as another type of reinforcement. Double-skinned FRP-steel CFFT members are also presented. A brief introduction to research on joints and splices of CFFT's is also included.

2.2 General Background of FRP tubes in structural applications

The use of FRP-wrapped concrete arose as a modification of steel-encased concrete columns, as FRP has a higher strength-to-weight ratio than steel. Columns with full jackets, discrete stirrups, or additional ribs made of FRP have been tested to determine the behavior of axially loaded FRP-confined concrete. Flexural tests of rectangular concrete beams encased in FRP and of hollow FRP tubes began the examination of the behavior of FRP in bending.

Fardis and Khalili (1981) tested FRP-encased concrete as concentrically loaded short circular columns. Adding one layer of FRP almost doubled the capacity of the concrete cylinder and increasing the number of layers incrementally made the cylinder stronger and increased deformation (see Figure 2.1). Failure of the cylinder occurred due to crushing of the concrete and failing the FRP simultaneously. Flexural behaviour of rectangular FRP-encased concrete beams with FRP box sections and an open top was also tested. FRP resists the tensile force at the bottom of the beam and is more effective than steel reinforcement since it is at the surface of the section compared to reinforcing bars, which are farther away due to cover requirements. Shear forces
along the sides of the beam were also resisted by FRP. The concrete core was the main compressive member and also provided protection against buckling of the thin FRP box. The FRP box formwork was designed to have closed ends to negate reliance on bond strength and to provide anchorage for the longitudinal tension fibres. The concept of ribs in the concrete or FRP for mechanical interlock was also presented. The beam with no longitudinal layers of FRP failed in a brittle manner through fracture of the woven FRP in tension. The beams with five or ten layers of FRP (considered over-reinforced) were more ductile and failed when concrete crushed in the compression zone, causing bursting of the two side walls of FRP near the top of the beam. Significant tension and shear cracks were seen in the beams after testing, although unloading led to the recovery of most of the deflection since the FRP was mainly intact. Deflections were larger than reinforced concrete beams with the same cross-section and typical steel reinforcement ratios due to the lower stiffness of the FRP, although the capacity of the FRP beam was higher and the strength-to-cost ratio was lower.

Nanni and Bradford (1995) tested 51 concrete cylinders jacketed with glass or aramid FRP. Cylinders failed due to tensile rupture of the jacket or at the overlap of the FRP sheets. Strength and pseudo-ductility were both increased but only showed improvement relative to a plain concrete cylinder after the unconfined strength of the concrete was attained. Models for predicting stress-strain behaviour of FRP-wrapped concrete were only effective at finding the strength of the cylinders, underestimating the ultimate strain.

Ibrahim (2000) tested hollow FRP poles with the intention for use as electrical transmission structures, due to their light weight and durability. A theoretical model was developed based on tests to predict the strength and deflection of hollow tubes in order to establish design guidelines. The twelve cantilevered poles were filament-wound and tapered over
their 6.25m length, from an inner diameter of 416mm at the base to 305mm at the top, and were fixed in a reinforced concrete footing. Tube thicknesses varied based on the number of layers (4-8), with longitudinal fibres oriented at ±5, ±10, and ±20° relative to the longitudinal axis; most of the poles (10 of them) had hoop fibres. Lateral deflections and ovalization were measured at the top, and strains were measured near the base. Ultimate load capacities were measured, and the poles were classified according to their load-to-weight ratio, using an existing classification system for wooden poles. Circumferential fibres and wall thickness were seen to be important in the capacity of the pole due to the local buckling restraint they provide. The experiment showed that GFRP poles are capable of achieving equivalent loads to wooden poles with one-third of the weight. Local buckling was the most common failure mode, occurring in all except one of the specimens, which failed in compression inside the concrete footing due to a lack of an internal stiffener.

These tests provide initial data on the behaviour of FRP-jacketed concrete columns undergoing axial compression. FRP as flexural reinforcement was also introduced on rectangular concrete beams. Tests on bending of hollow FRP tubes are used for development of theoretical design equations.

2.3 Flexural behaviour of CFFTs

After testing the behaviour of FRP-jacketed concrete cylinders in axial compression, tests increased on the flexural behaviour of this composite system. Test parameters include reinforcement ratio, fibre architecture, material type, concrete strength, magnitude of axial load, and presence of voids. Some tests attempt to reduce the weight of the system through the presence of central voids or only filling the tube near the support to gain the light weight and durability of the unfilled tubes discussed in the previous section.
Davol (1998) performed four-point bending flexural tests on 152mm diameter CFFTs with 85% of the fibres at ±10° relative to the longitudinal axis in the thin CFRP shell (2.29mm thick). The beam showed almost linear load-displacement behaviour with some slippage between the two materials and distributed concrete cracking. Failure occurred in compression, in the constant moment region. Another specimen with a thicker shell (4.57mm thick) displayed non-linear behaviour, but showed no slip between materials. Full-scale testing of CFFTs acting as bridge girders was also undertaken using larger tubes, with 345mm diameters and thicknesses of either 8.89mm or 9.65mm. Two of the shells were filled, two were hollow, and another was filled and overlaid with a concrete deck. Non-linear behaviour was observed and failure occurred in the constant moment zone in compression for the concrete-filled tubes. Strain profiles were almost linear throughout the cross-section, more so in the thinner shell as the thicker one had additional hoop fibres on the outside which may have warped. For the CFFT overlaid by the deck, 100 load cycles were applied until stiffness degradation became evident and shear connectors began to slip, and then it was loaded directly to failure.

Fourteen CFFTs were tested by Fam et al. (2003) in flexure with varying axial loads, including loads applied at an eccentricity. The effects of laminate structure and wall thickness on the interaction diagram are examined. Confinement plays a larger role in thin-walled tubes and varies depending on the concrete strength. In some cases, after reaching $f'_c$, the load dropped. For the tubes with higher stiffness in the hoop direction, the peak load was reached again and eventually the tube fractured. For the tubes with lower stiffness in the hoop direction, the load dropped more severely and the peak load was not reached again. For lower concrete strengths, it is typical to see a bilinear strain-hardening behaviour due to the onset of confinement before the load drops. Failure occurred for the beam specimens through tube tensile rupture in the constant
moment zone. The concentrically loaded columns fractured under biaxial stresses in axial compressive and hoop tensile stresses. Eccentrically loaded columns varied in their failure mode based on the eccentricity, through a tension rupture of the fibres, a balanced failure, or a compressive crushing of the fibres as the eccentricity decreased.

Mitchell (2008) tested both steel and FRP tubes that were partially concrete-filled along the length of the tube, as cantilevers, to determine the minimum filling length required to reach the ultimate moment of the tube. Partially filling the tubes allows the overall system weight to be reduced, which is preferable from a transportation or construction viewpoint. If the tube is not filled over a sufficient length, local buckling was observed to be the failure mode which is undesirable as it does not use the full material strength of the system. The optimal filling ratio, shown in Figure 2.2, would be the intersection of lines representing strength between beams that failed through local buckling of the tube and through tensile rupture of the tube. Mitchell (2008) tested six GFRP tubes with a diameter of 220mm with filling lengths ranging from 0% to 100%. It was found that the failure load and stiffness, each, almost doubled when compared to completely filled to hollow tubes (Figure 2.3). Partial confinement of the concrete was observed, as the strains in compression reached 0.01-0.015 (much greater than 0.0035). The neutral axis depth was observed to be at about 1/3 of the diameter from the top of the tube, for the tubes filled over more than half of their length. Filling the tube between 51% and 72% of the length showed similar behaviour and strength to the completely filled tube, while the 30% filled tube failed at a lower load by local buckling as seen in Figure 2.3. The optimal filling height was determined to be 34% of the tube’s length. A parametric study was also presented, and showed that the optimal filling ratio increases as the D/t ratio is reduced or the percentage of fibres oriented longitudinally increases.
Five beams were tested by Mirmiran (1999) – one in pure flexure, one in pure compression, and three with varying amounts of axial load. These beams were square with 178mm long sides, with 19mm thick transverse ribs spaced at 64mm centre-to-centre and 42mm wide longitudinal ribs in the centre of each side. The inner ply of bidirectional woven cloth was surrounded by 15 plies of ±75° filament wound fibreglass. The axial loads applied to the three combined loading specimens were based on compression tests of similar tubes without the inner woven layer. Failure for the pure bending specimen was gradual, starting with flexural and resin cracks and finally failing in tension at the peak load. The tube loaded in pure compression reached a peak load of over twice that of an unconfined column. Transverse and resin cracking were seen before the ultimate buckling failure and rupture of the tube near midspan. The beam with the lowest axial load failed through tension, and only one crack was seen in the concrete core when the tube was cut off. The other two beams with axial loads failed again in progressive rupture of the compression side of the tube, although it was less gradual than the other two beams. Based on strains from coupon tests, very little confinement occurred in the pure flexural specimen or beam with low axial loads. Confinement was only effective where compression failure occurred.

After testing specially made CFFTs with mechanical ribs for load transfer, an investigation into off-the-shelf circular tubes was done by Mirmiran (2000) to determine the cost-effectiveness and necessity of mechanical shear transfer. Sixteen 2.75m long specimens were fabricated from two different tubes (spun-cast or filament-wound). The spun-cast tubes were over-reinforced while the filament-wound ones were under-reinforced. They were completely filled with concrete and were loaded axially and then lateral loads were applied while keeping the axial load constant. The spun-cast tubes had a moment capacity of approximately twice that of the
filament-wound tubes, as seen in Figure 2.4. Ductile failure was observed under large deformation of the under-reinforced tubes. The spun-cast tubes typically failed through compression of the tube, and occasionally through longitudinal rupture of the tube. For the case of filament-wound tubes, failure was governed by tube fracture along its main winding axis at low axial loads while compression governed at high axial loads. Near failure, a plastic hinge was formed and midspan deflections increased under constant lateral loads. Some camber occurred in the members under axial loads due to the slenderness of the columns. Most deformation was recoverable, and little stiffness degradation was observed under repeated loading cycles, although the concrete had cracked and prevented complete recovery of deflections. Negligible slippage was observed in the axially-loaded columns, showing that the off-the-shelf products were acceptable as long as end conditions were properly detailed. Beam-column samples achieved similar loads to some conventional prestressed concrete columns. Large amounts of slip (50-150mm) were noted for the flexural specimens after failure at midspan, showing that mechanical interlock is necessary if no axial loads are applied. Lateral load capacity was seen to decrease as axial loads increased. Bilinear load-deflection behaviour was observed, with the change coming at the cracking of concrete. The spun-cast tubes had lower deflections and higher capacities.

Some effort has been put into exploring the limit between flexural and shear failure of CFFT s. Four types of GFRP tubes were tested by Ahmad (2004). Ten beams were tested in either three- or four-point bending. The reinforcement ratio of the sections has a large impact on the crack pattern of the beam – low reinforcement ratios have a few larger cracks while higher reinforcement ratios have flexural-shear and web-shear cracks as well as the flexural cracks. In deep beams, cracks are localized under the load point, while slender beams spread the cracks along the member. After flexural cracks develop, they spread quickly through the depth of the
beam, extending through the full depth, as large slip occurs. Therefore, the concrete cannot provide flexural resistance, and shear will not be more critical than flexure in short/deep CFFTs unless full composite action is developed. As the shear span-to-depth (a/D) ratio decreases, the ratio of longitudinal strain to shear strain approaches 1. If the ratio is less than 1, that means that shear strains are larger than longitudinal strains and so the beam is more susceptible to shear failure. As the a/D ratio decreased, capacity and slippage increase, while ductility decreases. Bernoulli beam theory does not apply for beams with an a/D ratio equal to 1, as seen by the fact that they all failed at a higher moment than slender beams. In deep beams, arching action and direct shear transfer through compression struts dominates.

These tests show typical failure modes for concrete-filled FRP tubes. Slip between the concrete and the FRP tube is shown to be an important factor that affects composite columns that do not experience axial loading. Tests by Ahmad (2004) helped to determine the minimum shear-span-to-depth ratio for flexural failure to occur, which influenced the choice of shear spans in the tests described in the following chapters of this thesis.

2.4 Flexural behaviour of reinforced/prestressed CFFTs

Testing of reinforced or prestressed CFFTs was an attempt to make CFFTs stronger and more ductile. The behaviour of reinforced and prestressed CFFTs was tested under axial and flexural loading conditions, and the influence of shear forces, slip, and fatigue on the strength and ductility of the system were investigated.

Cole (2005) tested seven CFFT beams, all 219mm in diameter, in four-point bending. Internal reinforcement varied in terms of both material (GFRP rods, CFRP rods, or steel bars) and reinforcement ratio. The confining reinforcement was also varied, comparing the GFRP tube to steel spiral reinforcement or to beams with no hoop reinforcement. Strength and stiffness are both
increased through the use of the FRP tube, which showed significant pseudo-ductility through the brittle failure of successive layers of the FRP. The column reinforced with a steel spiral also had more ductility than the unconfined concrete, although the failure strength was governed by the longitudinal reinforcement for the beams with spiral steel reinforcement and no hoop reinforcement. Using GFRP as the longitudinal rebar instead of steel did not change the strength when using an equivalent area of GFRP; however, the ductility was increased and the stiffness after cracking of the steel-reinforced beam was approximately twice that of the FRP-reinforced beam. Longitudinal CFRP rebar increased the strength relative to GFRP rebar as well as the stiffness, but both failed in a similar manner through longitudinal tube tension rupture, followed by rupture of the tension rebar. This type of failure is seen as a disadvantage due to the sudden nature of failure without adequate warning and ductility. Increasing the reinforcement ratio of steel rebar increased both the stiffness and strength of the beams. Using internal reinforcement for the beams provided enhanced crack control, leading to less slip between the tube and concrete core than unreinforced beams.

Cole (2005) cut two sections off the ends of each flexural beam and tested each in shear with a/D ratios of 1 or 2. All specimens with a/D ratios of 1 behaved like deep beams and failed in shear with diagonal tension cracks starting at the supports. Adding the FRP tube increased the strength in shear compared to the spiral-reinforced and unreinforced beams, by intercepting the shear cracks and providing hoop fibres which act as stirrups. Using FRP reinforcing bars decreased the shear strength compared to steel reinforcing bars because using the more ductile steel increased the dowel action. Beams with an a/D ratio of 2 failed in shear if they did not have a GFRP tube while all beams with a tube (except the one with CFRP reinforcing bars) failed in flexure. The beam with CFRP reinforcing bars underwent excessive slip between the concrete and
the reinforcing bars due to lack of bond with the concrete that did not allow the CFRP to achieve its full tensile strength. Slip was seen to be a critical factor which could change the failure mode from flexural to shear.

Mandal (2004) performed four-point bending tests on prestressed CFFTs (PCFFTs). Six specimens were tested to study the effect of prestressing level, tube laminate structure, reinforcement ratios, and pretensioning (as compared to unbonded post-tensioning). One of the prestressed beams was confined by steel spiral reinforcement. The capacity of the different methods was tested along with the stiffness degradation at various load levels through cyclic loading. Reinforcement levels were varied in both the FRP tube and steel, with three types of tubes being tested, with different ratios of longitudinal to hoop fibres, and different thicknesses. It was seen that prestressing activated the confinement earlier in the PCFFTs than in other CFFTs. PCFFTs also experienced no slip, which increased the capacity. Prestressing improved initial stiffness, cracking strength and flexural strength, but not the stiffness after yield, so that there was better energy absorption compared to CFFTs reinforced with conventional steel reinforcement. PCFFTs continued to gain load until rupture, while conventional concrete members with steel spirals were plastic once the longitudinal steel yielded. An increase in prestressing levels increases confinement. Increasing the thickness of the tube or, to a lesser extent, the percentage of longitudinal fibres, will increase the strength and stiffness. Post-tensioned unbonded beams were slightly less stiff after cracking and slightly lower in strength than pre-tensioned ones.

Gould and Harmon (2002) and Shao and Mirmiran (2004) investigated the behaviour of reinforced CFFTs under constant axial and cyclic lateral loads. Gould and Harmon (2002) tested twelve 180mm diameter CFFTs in cyclic flexure under constant axial loading, studying the effect of varying the axial load, fibre volume ratio, and column length. Loading was displacement
controlled, with three cycles at each displacement. Failure modes include fracture of the FRP, fracture of reinforcing steel, shear deformation of the column base, and overall column instability. Shear did not affect the moment-curvature behaviour of the columns but shear strains interacted with flexural and axial load-induced strains. All columns were ductile but became unstable at ratios required by seismic design, although it is important to note that all fibres were oriented in the hoop direction.

Shao and Mirmiran (2004) tested six 2.4m long tubes under constant axial and reverse lateral loads. Three of the tubes had the majority of the fibres in the longitudinal direction, while the other three had fibres oriented at ±55°. Of each type of tube, one specimen had no additional reinforcement while the other two had differing steel reinforcement ratios. The one with the longitudinal fibres failed in compression in a brittle manner, and the one with fibres at ±55° failed in a ductile tension failure. The main conclusion was that CFTTs can be as ductile as beams reinforced with steel bars if they have an appropriate laminate structure.

The ductility seen in reinforced and prestressed CFTTs is desirable for construction, although careful design of the tube laminate structure is required in areas of high seismic activity. The presence of internal steel reinforcement decreased the slip seen in the flexural tests on CFTTs which increased the strength of the specimens. The tests described in this section have a relatively low reinforcement ratio from steel, whereas CFFT-encased steel sections (as seen in this thesis) have a higher steel reinforcement ratio.

2.5 FRP-Steel-concrete composites

A double-skinned section, using an FRP outer tube and steel inner tube, reduces the weight through a central void, increases the stiffness with the steel tube, prevents buckling of the steel, and takes advantage of the compressive strength of the concrete. The low stiffness of the
FRP leads to higher deflections than a comparable steel tube. Combining the two types of tubes brings the light weight corrosion resistance, and confinement of the FRP, along with the ease of connection, ductility, and stiffness of the steel. This system was tested under axial and flexural loads. The double-skinned tube was modified to include a steel W-section as the interior member to maintain the increased strength and ductility with the high steel reinforcement ratio. Axial tests have been performed on this system with the test parameters including the FRP reinforcement ratio, FRP material (carbon compared to glass FRP), and columns shape.

Teng et al. (2004) tested FRP-concrete-steel double-skin tubular members as stub columns and beams in four-point bending. Three pairs of specimens were prepared, all with an outer diameter of 152.5 mm and a height of 305 mm but varying in the thickness of the tube. The steel tube had an outer diameter of 76.1 mm and a thickness of 3.2 mm. Along with these specimens, six cylinders were confined with one, two, or three layers of FRP and three samples of the hollow steel tube. Confinement was small for the single ply of FRP, while the capacity increased by 27% for two plies, and 48% for the three plies which is similar to concrete-filled FRP tubes, totally filled.

In a further stage of testing, Teng et al. (2004) constructed and tested three beams where the inner steel tube was moved closer to the tension side, as seen in Figure 2.5, to take full advantage of the material strengths. The specimens again had an outer diameter of 152.5 mm but were 1500 mm long. The beams had zero, one, or two layers of FRP. The beam without an FRP tube had large flexural and shear cracks along its length and its load descended post-peak. Both beams with FRP had large flexural cracks below the load points but did not fail before deflection limitations caused the test to end and were very ductile. Increasing the amount of FRP had only a
small influence on the capacity, while having an FRP in general increased both confinement and shear resistance over the beam without the FRP tube.

Yu (2006) tested the flexural behaviour of these hybrid FRP-concrete-steel double-skinned tubular columns in four-point bending and developed a theoretical model based on the plane sections assumption. The main parameters studied are the concrete strength and thicknesses of the FRP and steel tubes. Fourteen beams were tested in two phases. The first phase included only axi-symmetric specimens, where two beams had additional FRP reinforcing bars. The second phase involved eccentrically placed steel tubes, with variations in the concrete strength, FRP tube thickness, steel tube thickness, presence of FRP bars, and eccentricity of the steel tube. Behaviour was quite ductile because of the effect of the steel and the delayed concrete crushing because of confinement. Higher stiffness and strength were seen when the steel tube was shifted toward the tension side or when additional FRP reinforcing bars were added. The tests on several specimens with FRP tubes but no FRP reinforcing bars had to be terminated due to deflection limitations and instability. The thickness of the FRP tube had no effect on the stiffness and strength of the beams if the steel tube is concentric but impacts the ductility. The effect of confinement is significantly less for these beams than the columns due to the strain gradient in the cross-section and the smaller area of concrete in compression. For the eccentric steel tubes, ultimate load and ductility were both increased with thicker FRP tubes. Slip between the FRP tube and concrete was quite small due to the low axial stiffness of the FRP tube, but slip between the steel tube and concrete was much larger, about 10mm at the end. A sudden slip during the test was likely due to loss of composite action between the concrete and steel, so a method to increase bond strength between the steel and concrete is recommended, as well as to increase the bond strength between the concrete and FRP tube, especially if the tube has a higher axial stiffness.
Wong (2008) tested 43 specimens with a diameter of 152.5mm in axial compression, grouped into 18 double-skinned tubular columns, 11 FRP-confined solid cylinders, and 14 FRP-confined hollow cylinders. Parameters studied include the effect of outer FRP tubes and inner steel tubes on the behaviour of the columns, the effect of concrete strength, the effect of the void ratio (the ratio of the inner diameter to outer diameter of the hollow concrete section) on FRP-confined concrete, both with a steel tube and with a void but without an inner tube. All solid and double-skinned specimens failed through rupture of the outer FRP tube. Cylinders with more plies of FRP tube were more likely to buckle, as were cylinders inner with steel tubes, with higher D/t ratios. The size of the void affected the failure of the specimens, as the ones with larger voids reached peak stress prior to rupture, while the ones with the smaller voids continuously increased in stress until tube failure. The cylinders with larger void ratios had larger axial strains than the equivalent solid cylinders. Hollow and double-skinned tubes were similar in behaviour and confinement, but differed from the solid cylinders because of the unequal confining pressures generated.

Karimi (2010) performed axial tests on a circular CFFT-encased steel w-section as short columns. The steel on its own was tested as a column to compare the results to previous tests on unreinforced CFFTs. A shrinkage reducing chemical was added to the concrete mix of one specimen for the tests to compare the effect of shrinkage. Some tubes had a splice mechanism for the tube to test its effectiveness. An analytical model, including a simplified approach, was created.

Seven columns were tested by Karimi (2010). Three of the columns were the bare inner steel sections, to test the enhancement in capacity. The fibres in the GFRP tubes were unidirectional circumferential, and had an outer diameter of approximately 220mm (there were
two types). The steel sections were W150x14 with yield strengths of 411MPa, and the concrete had a compressive strength of 48.3MPa. Failure of the columns occurred through rupture of the tube and immediate spalling of the concrete core. Local buckling of the steel flanges and web were seen in all columns after the tube was removed. Increased energy dissipation was evident from the increased load and increased failure axial strain.

Karimi (2011) then tested columns consisting of an I-beam wrapped with a carbon-FRP (CFRP) jacket in a rectangular shape with concrete filling the voids, as seen in Figure 2.6. To reduce stress concentrations at the corner of the cross-section and increase confinement, steel reinforcing bars were welded to the corners and then the beam was wrapped and filled with concrete. The jacket is expected to increase the axial capacity by adding strength from the confined concrete and preventing outward buckling of the flanges. The stiffness and energy dissipation characteristics are also expected to be improved. Appropriate detailing for load transfer is a key issue in construction. The main variables in this study were the number of CFRP wraps and corner radius of the wraps. Seven columns were tested, two as control and five with one to three layers of CFRP. A layer of GFRP was placed around the steel section first to prevent galvanic corrosion. The FRPs were unidirectional with all fibres running in the hoop direction to provide maximum confinement. Failure of the composite columns all occurred with rupture of the FRP at the corners, and local buckling of the steel and crushed concrete were seen after unwrapping the specimens. Increasing the number of wraps increased the capacity of the columns but had less effect on deflection and very little impact on stiffness. Increasing the corner radius also increased the compressive strength.

Similar to reinforced or prestressed CFFT's, double-skin tubes with FRP surrounding a hollow steel section provides a significant increase in strength and ductility compared to a plain
CFFT. A W-section encased in a CFFT shows similar axial behavior to the hollow sections. This thesis will investigate the flexural behaviour and a potential connection of this W-section encased in a CFFT.

2.6 Connections of CFFTs

Previous research on connections of CFFTs can be divided into two main groups: joints between members and splices within the same member. Chapter 4 of this thesis relates directly to the impact that the type of connection has on the strength of the CFFT. Selected research on splices is summarized in this section as similar methods can be used to connect a CFFT to the rest of the structure, but the majority of the relevant work relates to connections between CFFTs and other members.

2.6.1 Joints

The joints presented in this section are typically between unreinforced CFFTs and a reinforced concrete footing, although steel reinforcing bars are also used to create a more effective load transfer mechanism to the footing.

Lai (2010) tested two types of moment-resisting connections between an unreinforced CFFT and a reinforced concrete footing, one with direct embedment of the precast CFFT into the footing, and in the other the tube was adhesively bonded to a steel-reinforced concrete stub protruding from the top of the footing, then was filled with concrete. For both connections, the length of the connection was varied to determine the minimum embedment length required to achieve the strength of the CFFT members. The bond strength between the outer surface of the tube and the footing was tested in push-through tests and found to be 0.71-0.79 MPa. Additional low cycle bending fatigue tests to examine ductility of the plastic hinge were carried out.
In Lai’s (2010) first set of five cantilever tests on 219mm diameter tubes with the first type of connection, failure varied between flexural failure (tensile rupture) and excessive slip between the CFFT and the footing. The two tests that failed at the maximum load through tensile rupture, as seen in Figure 2.7, showed pseudo-ductility through progressive slip between the footing and the CFFT after radial cracks occurred in the footing. When the specimens failed through excessive slip, radial cracks in the footing seen in Figure 2.7 were much larger and the strains were significantly lower than the cases that failed through tensile rupture. The optimal embedment length of an unreinforced precast CFFT into a reinforced concrete footing was found to be 0.73D, where D is the outer diameter of the tube.

In Lai’s (2010) second set of tests, which used 169mm diameter tubes, a short reinforced concrete stub was cast as part of the footing, to provide a connection method that would not require shoring for the cast-in-place CFFT. The ideal failure for this connection is again tensile rupture of the CFFT to develop its full strength, but the secondary failure is bond failure of the adhesive between the FRP tube and concrete stub. Eight cantilever specimens were tested, including four with varying stub lengths, two with different steel reinforcement ratios in the stub, and two were tested in low-cycle fatigue. There was a significant difference in the deflection within the short stub relative to that of the portion of the CFFT beyond the stub. Low strains were noted adjacent to the footings due to the tube pulling away. The optimal length of the stub was found to be 1.1D, and the minimum reinforcement ratio of the stub to achieve tensile failure of the tube was 3.4%.

Zhu (2004) considered four joints: splices, beam to column connections, column to RC pier caps, and column to RC footings. Connections were either post-tensioned, dowel reinforced, surface bonded as a male-female joint, or embedded (into the RC concrete), as seen in Figure 2.8.
The pile cap was a stay-in-place form also made of FRP sheets and attached with an extra sheet on top for negative moment continuity. Joint stiffness was significantly lower than stiffness of the CFFT, so it behaved as a rigid body. Post-tensioning was the best method as it was both the strongest and most ductile. Male-female joints were not sufficient to transfer loads. Internal reinforcement outside of the connection was not necessary to achieve the full strength of the members.

Zhu (2004) also connected CFFT columns to an RC footing and compared the specimens to a control RC column connected to the same footing. Two of the columns were precast, one with dowels and the other being post-tensioned to the footing. The other was cast-in-place with dowels extending from the footing. Cyclic loading based on the deflection control was applied. Some separation between the FRP tube and footing was seen at high displacements but load was still transferred effectively, and a plastic hinge formed. Some matrix cracking was seen in the FRP tube of the cast-in-place system, but no fibres ruptured. All three joint methods performed in a very similar manner, and were much more ductile than the control RC column. Better performance was explained by the confinement of the concrete by the tube, along with a small contribution from the embedment in the footing. Post-tensioning was not particularly effective compared to the other systems but had the effect of reducing residual deformations.

Chapter 4 presents further research done using the moment-resisting connection by direct embedment into the concrete footing, specifically with regard to the slip between the CFFT and the concrete footing. The joints with steel connections between the CFFT and reinforced concrete footing show that steel has the effect of reducing slip between the CFFT and the reinforced concrete footing.
2.6.2 Splices

Most of the splice methods involve connecting the two pieces of CFFT using steel plates embedded in the concrete or the existing reinforcing bars, although a method is described which connects the two pieces using the exterior FRP shell.

Zhu (2006) studied the feasibility of four field splicing techniques. Three internally spliced beams were tested with grouted steel bars, grouted FRP bars, and unbonded post-tensioning bars. The fourth technique was an FRP socket, typical of the pipe industry. These were compared to a plain CFFT that was not spliced over its length. The spliced beams were initially stiffer due to the contribution of the steel, but did not reach the same capacity of the unreinforced CFFT. Grout strength was a contributing factor, along with the obvious issue of tube continuity. A longer socket or threaded sleeve is suggested as a possible method of providing continuity of the FRP in a splice, as is combining internal and external splicing, but neither option was tested. The control beam had end slip of 15.2 mm at one end, although composite action was achieved at midspan. Failure of the two bar-spliced beams was similar in that failure was determined based on excessive bar slippage and joint opening, but with more sharp load drops. The socket capacity depended on the shear strength of the epoxy, as the behaviour was linear-elastic until the epoxy started cracking.

Helmi et al. (2005) tested the driving ability of whole and spliced CFFT piles. Four piles of 357mm diameter and 13.7m length were driven and then extracted to examine for any damage. The extracted piles were then tested in bending. Three control specimens and two spliced control specimens were also fabricated but not driven to compare. The splice consisted of two steel plates with threaded rod connecting the two plates as shown in Figure 2.10. Each plate had four small T-
shaped grooves around the outside, which were matched and then connected with an I-shaped key. This connection was 7% stronger than the unspliced tubes when tested, which can be partially attributed to the presence of the rebar used to connect the plates.

When the piles were cut, there were three unspliced 6m specimens to be tested in four-point bending with a 5m span and 1m constant moment zone. For the three spliced 6m specimens, they were tested in three-point bending with a 4.5m span. The splice was positioned 550mm from the loading point, under a combination of high shear and moment. The remaining sections of the beam were cut into 300mm long sections to do push-off tests, to determine the deterioration of the bond strength at the top, bottom, and middle of the driven piles. The load was applied to the tube at the top of the specimen, and the concrete was supported at the bottom such that the concrete core could move relative to the tube.

The piles were seen to have cracked during driving, but only had a 5% reduction in their capacity with the reduction being a bit higher at the top of the pile. Bending stiffness was not much affected by driving. Very little slip was noticed for any of the beams, showing there is still good composite action even after driving. Bond strength was helped by an internal wavy pattern of the tube surface after the concrete-FRP adhesion was overcome.

Some successful splicing methods are described above, with slip between the concrete and steel connection, and continuity over the splice length being major factors in the strength of the connection. The splices which had a significantly decreased strength compared to the capacity of the unspliced member may be sufficient, depending on the location of the splice and distribution of the load along the member.
Figure 2.1. Axial stress-strain plots of FRP-jacketed concrete cylinders (Fardis and Khalili, 1981)

Figure 2.2. Determination of optimal concrete filling length ratio for partially filled CFTTs (Mitchell, 2008)
Figure 2.3. Load-deflection behaviour of partially filled CFFTs (Mitchell, 2008)

Figure 2.4. Moment-deflection curves for beams under no axial load (Mirmiran, 2000)
Figure 2.5. Cross-section of beams as tested by Teng et al. (2004)

Figure 2.6. Cross-sections of FRP-wrapped steel-concrete-I-beams, with and without corner treatment (Karimi, 2011)
Figure 2.7. CFFT tensile rupture and excessive slip/bond failure of CFTs embedded into a reinforced concrete footing (Lai, 2010)

Figure 2.8. Reinforced concrete footing joint detail for precast CFFT and post-tensioned CFFT columns (Zhu, 2006)
Figure 2.9. Mid-span confining strains in CFFT (Zhu, 2004)

(a) End plates
(b) Bars screwed into plates

Figure 2.10. Splice details (Helmi et al., 2005)
Chapter 3

Flexural Performance and Moment Connection of Concrete-Filled GFRP Tube-Encased Steel I-Sections

This chapter introduces a new system where a structural steel I-section is encased in the CFFT member for enhanced strength, stiffness and ductility. At the same time, the CFFT member provides support to the steel section, preventing local buckling, which allows its plastic capacity to be achieved. The steel section also facilitates a moment connection between the CFFT member and footings or beams, through a welded steel base plate. It can be argued that the FRP tube protects the concrete core, including the steel section, from moisture intrusion and potential corrosion.

This study addresses the flexural performance of the hybrid system, through beam tests, and the performance of the moment connection, through cantilever tests. A model is also developed to predict the moment capacity of the connection, while the flexural behaviour of the member is modeled using the computer program Response 2000.

3.1 Experimental Program

This section describes the materials used, namely the GFRP tubes, steel I-sections, and concrete in addition to the test specimens and parameters studied. The fabrication process for both the bending and moment connection tests is listed, including the test setup and the instrumentation.

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1 Most of this chapter has been submitted as the following journal paper: Zakaib, S., and Fam, A. (2012). “Flexural Performance and Moment Connection of Concrete-Filled GFRP Tube-Encased Steel I-Sections”, ASCE Journal of Composites for Construction, 16(5), pp. 604-613.
3.1.1 Materials

3.1.1.1 GFRP Tubes:

Four different types of filament-wound glass/epoxy FRP tubes, designated as T1 to T4, were used in this study. T1 and T2 had similar outer diameters of 169 mm and similar fibre architecture, but different wall thicknesses of 3.55 and 5.19 mm, respectively. The fibres were provided in nine alternating layers oriented longitudinally (+9° in T1 and +6° in T2), and circumferentially (-86° in T1 and -85° in T2), where the angles are measured from the longitudinal axis of the tube. T3 and T4 tubes had similar outer diameters, 114 and 115 mm, respectively, and differed primarily in fibre architecture in addition to a slight difference in wall thickness. T3 tubes had ten alternating layers oriented in the longitudinal (+6°) and hoop (-85°) directions, while T4 had a ±55° eight-layer angle-ply laminate. The wall thicknesses of T3 and T4 were 4.01 and 5.07 mm, respectively.

Tensile testing of longitudinal GFRP coupons was undertaken according to a slightly modified ASTM D3039 standard, using three coupons for each tube. The coupons were 25 mm wide and 255 mm long, including a 15 mm central region between gripping tabs. Each end had a 120 mm long portion prepared for gripping using wet epoxy lay-up of fibreglass fabric. The roundness of the tube was accounted for by flattening the tube with weights as the fibreglass tabs were hardening. Figure 3.1 shows the stress-strain curves of all four GFRP tubes, with the stress being calculated as the measured force from the coupon tests divided by the cross-sectional area of the flattened tube and the strain being the averaged value of the strain measured by a strain gauge on either side of the coupon. A somewhat bilinear behaviour is observed for tubes T1 to T3 with changes in the modulus occurring between 0.005 and 0.01 strains, when the hoop layers split. For T4, after the initial elastic range, the stress-strain curve gradually transitioned to a flat
plateau, similar to yielding in steel, due to the laminate architecture. The ultimate strength of T4 tube at failure was significantly lower than other tubes because failure occurred by splitting of the fibres in a V-shaped pattern, and not by longitudinal fibres fracture as in the other tubes. The average ultimate strengths of coupons from tubes T1 to T4 were 168, 187, 202 and 53 MPa, while the average ultimate strains were 0.0186, 0.0189, 0.0198 and 0.0184, respectively.

3.1.1.2 Steel Sections:

Two I-shaped steel S-sections, referred to as S1 and S2, were used. S1 had a depth of 102.1 mm, a flange width of 68.2 mm, a web thickness of 5.57 mm and a tapered flange, varying from 10.01 to 5.94 mm in thickness. S2 had a depth of 76.0 mm; a flange width of 62.5 mm, a web thickness of 5.97 mm, and a tapered flange, varying from 5.81 to 5 mm in thickness.

Tensile testing of steel coupons was carried out according to ASTM E8 using three 200x20 mm dog-bone coupons for each section with a 75x12.5 mm gauge length, cut from the web since the flanges were tapered. Figure 3.2 shows the stress-strain curves for the two types of steel. The average yield and ultimate strengths for S1 were 400 and 475 MPa, and for S2 were 370 and 460 MPa, respectively. The modulus of elasticity for S1 was 195GPa, and for S2 it was 200GPa.

3.1.1.3 Concrete:

A typical concrete mix with maximum aggregate size of 9.5 mm was used. Compressive tests of 100x200 mm cylinders were conducted according to ASTM C39-96. The compressive strengths based on the average of three cylinders for each test specimen ranged from 32 to 38 MPa on the day of testing.
3.1.2 Test Specimens and Parameters

The experimental study comprises two phases. Phase I addresses the flexural behaviour, strength, and failure modes of beam specimens of the hybrid system. Ten beams were tested to failure in four-point bending (see Table 3.1(a) for details). The key parameters examined were GFRP tube thickness, fibre architecture of the tube, and reinforcement index ($\omega$). The reinforcement index is the reinforcement ratio normalized with respect to the materials’ relative strengths, given the hybrid nature of reinforcement, and is calculated as in Equation [3.1]:

$$\omega = \frac{4t f_{tu}}{D f'_c} + \frac{4A_s f_y}{\pi D^2 f'_c}$$

where $D$, $t$ and $f_{tu}$ are the average diameter, wall thickness, and longitudinal tensile strength of the tube, respectively. $A_s$ and $f_y$ are the steel cross-sectional area and yield strength, respectively. The concrete compressive strength is $f'_c$. A summary of reinforcement index values is provided in Table 3.1(a) for all specimens except those using tube FT4. Tube FT4 is an angle-ply tube, so the tube does not provide direct longitudinal reinforcement. Also, the coupons for tube FT4 did not fail through tensile rupture so the value for longitudinal tensile strength is unknown. Specimens CFFT1 through CFFT4 were concrete-filled tubes without steel sections, while their counterparts CFFT1-S1 to CFFT4-S2 included the embedded steel sections as seen in Figure 3.3. Steel specimens S1 and S2 were the control bare steel I-section beams.

After studying the general flexural behaviour of this system in Phase I, a moment connection for CFFT members, utilizing this hybrid system, was developed and tested in Phase II. The configuration of CFFT3-S2 from Phase I was selected for this study. A 330x330x19 mm steel base plate, envisioned to be bolted to another structural member such as a concrete footing or a steel girder, was welded at the end of the S2 steel section. To examine this moment connection, the CFFT-encased steel section was tested using a cantilever bending setup. Five
specimens were tested as described in Table 3.1(b), where the length of the steel section ($L_s$) was varied inside the CFFT member from 80 to 800 mm (i.e. 0.10 to 1.0$L$, where $L$ is the span), which can be seen in Figure 3.4. The objective was to establish the minimum embedment length required for the CFFT member to develop its full flexural strength as well as the minimum embedment length required to develop the full capacity of the plastic hinge at the fixed end.

3.1.3 Fabrication

For beam specimens CFFT1, CFFT1-S1, CFFT2 and CFFT2-S1, in Phase I, the GFRP tubes and associated steel sections, were cut to 1.3 m lengths, while those for specimens CFFT3, CFFT3-S2, CFFT4 and CFFT4-S2 were cut to 0.9 m lengths. The tubes were cleaned and placed upright. Both the tubes and steel sections were secured in place at the bottom by small wooden pieces nailed to a flat plywood plate. At the top, steel sections were secured in place during the concrete pour by shimming against the tube as shown in Figure 3.3, and then shims were removed and the concrete was filled flush with the top of the tube. Concrete was vibrated to ensure complete filling of space between the steel section and the tube.

For cantilever specimens in Phase II, the steel sections of various lengths were groove-welded to the 330x330x19 mm steel plates seen in Figure 3.4. The plates were rested on the floor and the tubes were fitted around the steel sections such that they sat flush on the steel plates, and were held in position at the bottom by small angles. Concrete was then poured and consolidated by a steel rod.

3.1.4 Test Setup and Instrumentation

All specimens in Phase I were tested as simply supported beams in four-point bending. Specimens CFFT1 to CFFT2-S1 had a span of 1180 mm and a constant moment zone of 340 mm as shown in Figure 3.5, while CFFT3 to CFFT4-S2 as well as S1 and S2 had a span of 800 mm
and a constant moment zone of 230 mm. Dimensions of the tube and test setup are listed in Table 3.1(a). Narrow steel channel sections, with plaster fill, were used at the loading and support points to conform to the round surface of the tubes. Rollers were used at loading and support points to allow for small displacements. A loading rate of 1 mm/min was used.

All specimens in Phase II were tested as cantilevers with a 730 mm span measured from loading point to fixed end. To provide the end fixity, the end steel plates were connected to a rigid steel assembly using four bolts at the corners of the plates as seen in Figure 3.7. The steel assembly was clamped to the base of the testing machine using high strength threaded steel rods. The load was applied using a knife-edge to a narrow channel section sitting on the tube, with plaster fill, to conform to the round surface. The channel had brackets welded on either side of the knife-edge to prevent the loading point from slipping off the channel. A 1 mm/min rate of loading was used.

Deflections were measured at mid span in Phase I and under the loading point in Phase II, using linear potentiometers (LPs). Longitudinal strains were measured using electric resistance strain gauges and displacement-type PI gauges, at various locations on the steel section and tube. Relative slip at the ends between the tube and concrete core in Phase I, and the tension gap opening between the end of the tube and steel plate in Phase II, were measured using LPs. A load cell built into the 900 kN testing machine was used to measure loads.

3.2 Experimental Results

3.2.1 Phase I: Beam Specimens

Figure 3.9Figure 3.10 show the load-deflection and load-longitudinal strain responses of specimens CFFT1, CFFT2, CFFT1-S1, CFFT2-S1, and S1, while Figure 3.11Figure 3.12 show similar responses for specimens CFFT3, CFFT4, CFFT3-S2, CFFT4-S2, and S2. Since the plain
steel beam S1 was much shorter than the rest of the beams in Figure 3.9 and Figure 3.10 (800 mm compared to 1180mm), its loads and deflections were normalized to an equivalent span of 1180 mm for comparison to CFFT1-S1 and CFFT2-S1. Figure 3.13 shows the load-slip responses of the specimens created using the larger diameter tubes (FT1 and FT2), showing a decrease in the amount of slip for the tubes with the steel inside. The smaller tubes (FT3 and FT4) did not have sufficient space for the instrumentation required to measure slip between the tube and concrete, although visual observations showed a similar decrease in slip with the addition of a steel section. Figure 3.14 shows the variation of neutral axis depth with load. Figure 3.15 through 3.17 show the failure modes.

3.2.1.1 Tubes with Cross-Ply Laminates:

Specimens CFFT1 and CFFT2, without steel sections, had somewhat similar laminate structures but the wall thickness was slightly larger for CFFT2. As such, CFFT2 showed a slightly higher strength and stiffness relative to CFFT1 as shown in Figure 3.9. The flexural responses for both specimens are relatively linear beyond cracking of the concrete, up to failure which occurred by tensile fracture of the tube as seen in Figure 3.15. End slip between the concrete core and the tube developed gradually beyond cracking of the concrete, reaching a maximum of 1 and 2 mm for CFFT2 and CFFT1, respectively (Figure 3.13). Figure 3.14 shows that neutral axis depth is generally quite stable for CFFT's beyond cracking, at about 0.3 of the diameter. All these observations are quite consistent with those reported in literature for CFFT's with cross-ply laminates of fibers oriented close to the longitudinal and circumferential directions (Fam and Rizkalla, 2002).

Adding the steel sections changed the flexural performance of CFFT's with cross-ply laminates drastically, as evidenced by specimens CFFT1-S1 and CFFT2-S1, also shown in Figure
3.9. A bilinear response with significantly higher strength and stiffness, relative to their plain CFFT counterparts, can be seen. The cracking load also increased to 1.5 to 2 times that of CFFTs without steel sections. Once the tubes fractured in tension, as seen in Figure 3.15, the load dropped to about two thirds of the peak strength and remained stable over a large range of deflection, until the tube crushed in compression with some evidence of tensile fracture in the circumferential direction, shown in Figure 3.16. This behaviour is similar to that of CFFTs with internal longitudinal steel rebar (Cole and Fam, 2006). Figure 3.13 shows that the presence of the steel section resulted in reduced end slip between the concrete core and tube. It also shifted the neutral axis depth to about 0.45 the diameter, which also remained stable throughout the loading history (Figure 3.14).

3.2.1.2 Tubes with Angle-Ply Laminates:

Specimens CFFT3 and CFFT4, without steel sections, had somewhat similar dimensions but completely different laminate structures, namely cross-ply and angle-ply, respectively. As a result, the load-deflection response of CFFT4 in Figure 3.11 is much more ductile than that of CFFT3. This is consistent with the coupons results in Figure 3.1. CFFT3 failed by tensile fracture of the tube, very similar to CFFT1 and CFFT2 seen in Figure 3.15, however, CFFT4 deflected significantly until the test became unstable although the tube did not reach failure.

Adding the steel section in specimen CFFT4-S2 with angle-ply laminate showed a significantly different response from CFFT3-S2 with cross-ply laminate seen in Figure 3.11. In CFFT3-S2, the behaviour was quite similar to CFFT1-S1 and CFFT2-S1 in that the load dropped upon tension failure of the tube to a stable level and remained sustained until the tube failed in compression as shown in Figure 3.16. In CFFT4-S2, the load and stiffness increased significantly because of the steel section. The load, however, remained stable at the peak level without drop,
until the test was terminated due to excessive deflection. It is clear that in this type of angle-ply tube, the steel section enhances the strength and stiffness but not ductility as the CFFT's are quite ductile on their own.

3.2.1.3 Effect of Reinforcement Index:

Given the hybrid nature of reinforcement in these members, a reinforcement index ($\omega$) is used to quantify the steel and GFRP reinforcement in tubes failing in tension, as discussed earlier and summarized in Table 3.1(a) based on Equation [3.1]. To study the effect of the reinforcement ratio on flexural strength, Figure 3.18 shows the variation of the ultimate moment, $M_u$, of the test specimens normalized with respect to the diameter and concrete strength. Specimens CFFT4 and CFFT4-S2 did not fail in tension by fracture of the tube, but rather by excessive deformation. Also, coupons from their tube (T4) failed by splitting and sliding of the fibres seen in Figure 3.1 and not by tensile fracture of fibers. As such, the ultimate strength, $f_u$, of this tube is not exactly known; therefore, specimens CFFT4 and CFFT4-S2 were not included in Figure 3.18. Figure 3.18 clearly shows a distinct trend; with data points at the lower end representing CFFT members, while those at the higher end representing CFFT-steel hybrid members. This correlation of normalized parameters may be used as guide to assist designers, however, it is noted that only a few data points are available.

3.2.2 Phase II: Cantilever Specimens

Figure 3.19 and Figure 3.20 show the load-deflection and load-longitudinal strain responses of cantilever specimens CS1 to CS5, where the strains are measured at various locations on the GFRP tube and steel section. Figure 3.21 shows the load-tension gap responses, where the measured gap is located between the fixed steel plate and tube and represents the effect
of strain of the steel section and/or slip between the steel section and concrete. Failure modes are shown in Figure 3.22 through Figure 3.24.

3.2.2.1 Effect of Length of Steel Section:

Specimens CS1 and CS2 with \((L_s/L)\) ratios of 0.10 and 0.192 had a bond-slip failure seen in Figure 3.22, where the short steel sections slipped gradually from the concrete cores of the CFFTs. As such, the load-deflection responses are characterized by an increasing trend until the peak loads are reached and then the load drops, more gradually in CS1 than CS2. It was also observed that the tube fractured in the hoop direction on the compression side at the fixed end as shown in Figure 3.22. Figure 3.21 shows the gradual increase in tension gap for CS1 and CS2, up to about 17 and 13 mm, respectively, which is primarily slip in this case, since the steel sections never yielded, as evident from the strains measured in Figure 3.20(a) and (b). It is worth noting that the cross-sections of the CFFT members at the end of the steel sections show tensile strains in the tube lower than the rupture strains of the tube.

In specimen CS3 with \((L_s/L)\) ratio of 0.315, the steel section was long enough such that global bond failure did not occur. At the same time, it was not long enough to develop the plastic capacity at the fixed end as the steel has barely yielded as seen in Figure 3.20(c). In this case, the critical section that governed failure was in fact just at the end of the steel section, within the CFFT member, where the flexural tension failure seen in Figure 3.24 occurred by fracture of the tube. The tube tensile strain in Figure 3.20(c) at this section shows a lower strain value than failure strain. It is likely that the strain gauge did not survive to the end and malfunctioned. The flexural behaviour of CS3 is relatively linear because of this brittle failure (Figure 3.19). Figure 3.21 shows that the maximum tension gap measured for specimen CS3 was approximately 4 mm. Since the steel barely yielded, this gap reflects primarily some slip between the steel and concrete.
core and possibly some slip between the core and the tube. As such, the moment at the failed cross-section was about 20 percent lower than that of specimen CFFT3 in Phase I, which had the same GFRP tube, but a different concrete mix.

Specimens CS4 and CS5 with \((L_s/L)\) ratios of 0.552 and 1.0 were able to yield and develop a full plastic capacity as shown by their load-deflections responses (Figure 3.19). Loads reached a peak value after excessive deflection and the load dropped slightly afterwards but continued to be sustained over very large deflections, until unloading was commenced due to excessive deflection. Figure 3.22 shows the large tension gap opening near peak load. Figure 3.21 shows the development of this gap with load. Before yielding, the gap is quite small, less than 2 mm. Once the steel yields, the gap opening increases, reflecting elongation of the steel section, accommodated by localized slip of the steel section from the adjacent concrete. In specimen CS4, the CFFT cross-section at the end of the steel section was not critical as the tube strains were less than fracture strains.

The strain in the steel is relatively constant over the length of the steel section for CS3, CS4, and CS5 in Figure 3.20(c), (d), and (e). For CS3 and CS4, this is consistent with visual observations during the test that the stiffer section of the cantilever (containing the steel beam) appeared to deflect as a rigid body, while the change in curvature along the member was more visible in the section of the beam beyond the steel section. For CS4 and CS5, the strain in the tube at a distance of \(D\) from the fixed end was close to the strain in the steel, as the whole section was acting as a rigid member until cracks in the concrete and FRP caused the section to behave non-compositely.
3.2.2.2 Failure Envelope:

Figure 3.25 shows the variation of maximum moment achieved at the fixed end with (L_s/L) ratio. Also shown is the failure envelope predicted by an analytical model, described later. Three distinct regions can be observed. In the first region, as the embedment length of the steel member increases, the moment capacity increases proportionally and failure is governed by bond between the steel section and concrete core. In the second region, the strength also increases as the steel member increases in length, but in this case, flexural failure occurs in the CFFT member at the free end of the steel member. The third region is somewhat flat and represents the ceiling of strength based on plastic capacity. Based on the average capacities of CS4 and CS5 specimens, it can be seen that the plastic hinge capacity (17 kNm) is only about 20% of the capacity of the hybrid CFFT-steel member as measured in four-point bending in Phase I, which is specimen CFFT3-S2 in this case. This is because the tube is discontinuous and therefore ineffective in tension at the fixed end moment connection.

Two distinct critical (L_s/L) ratios can be observed in Figure 3.25, one represents the transition from bond failure to CFFT flexural failure, and the other represents the transition from CFFT failure to the full plastic capacity. An analytical model is developed and presented next, in Section 3.3, to predict the full failure envelope, including the two critical (L_s/L) ratios.

3.3 Analytical Modeling

3.3.1 Moment Connection Models

This section describes design-oriented simplified models used to predict the ultimate moment capacity of the CFFT-encased steel section based on the moment connection described earlier. The model can establish the failure envelope illustrated in Figure 3.25, for the full range of (L_s/L) ratios. The failure envelope can be characterized by three distinct regions, namely, a
bond failure region at the connection at small \((L_s/L)\) ratios, a CFFT flexural failure at the end of steel section for moderate \((L_s/L)\) ratios, and a plastic hinge flexural capacity at the fixed end for large \((L_s/L)\) ratios. A model is described next for each of the three regions.

### 3.3.2 Bond Failure Capacity

At small \((L_s/L)\) ratios, excessive slip between the steel section and concrete core occurs. Figure 3.26 shows a free body diagram simplification of this case, where a force couple is developed as the tube pivots around a bearing point of the tube against the steel plate, while the steel section slips off the concrete core. Bond stresses develop over the surface area of the steel section. The integral of these stresses provides the tension force in the steel section that balances the compression force at the bearing point. Park and Pauley (1975) reported the bond strength \((\tau_u)\) as a ratio of the concrete cube strength \((f_{\text{cube}})\), for smooth uncorroded steel bars embedded vertically in concrete. The reported bond-slip response is presented in Figure 3.27. In the current study, it is assumed that the bond behaviour of a steel I-section is somewhat similar to that of smooth steel bars. Figure 3.27 shows that \(\tau_u\) remains constant, at 0.088 \(f_{\text{cube}}\), over a large range of slip. As such, a uniform value of \(\tau_u\) is assumed over the surface area of the steel section, over its full depth, despite that fact that slip at the level of the top steel flange is larger than the bottom flange. In the experimental study the concrete compressive strength based on the average of cylinder tests is 35 MPa, and assuming that cylinder strength is 0.8 times the cube strength, is established as 3.8 MPa. Taking moments about the bearing point of the integrals of over the surface areas of the steel flanges and the web, shown in Figure 3.26, gives the ultimate moment \(M_u\) for various \((L_s/L)\) ratios, within the region of bond failure. For the specimens tested in Phase II of the experimental program which experienced bond failure (CS1 and CS2), this model shows very good predictions (Figure 3.25). Detailed calculations are given in Appendix A.
3.3.3 CFFT Flexural Failure

Beyond a critical \( (L_s/L) \) ratio, referred to herein as \( (L_s/L)_{cr,a} \) ratio, bond failure does not become critical and a cross-section of the CFFT member at the end of the steel section (section (a-a) in Figure 3.28) controls failure and the strength of the system. For CFFTs failing in flexure by rupture of the tube, Fam and Son (2008) proposed the following equation for the ultimate moment \( M_u \) of a CFFT:

\[
M_{u, CFFT} = 0.0045D_0^3 f_t' \left( \frac{100}{D_0} \frac{f_{tu}}{f_t'} \right)^{0.815}
\]  

[3.2]

where \( f_{tu} \) is the ultimate tensile strength of the tube, \( t \) is the tube thickness, and \( D_0 \) is the outer diameter. Knowing \( M_u \) CFFT at the end of \( L_s \), for a given bending moment diagram, the corresponding fixed end moment at failure (\( M_u \)) can be established. For a triangular moment distribution as in the experimental study:

\[
M_u = M_{u, CFFT} \left( \frac{L}{L - L_s} \right)
\]  

[3.3]

For specimen CS3 that had a CFFT failure, the model gave a good prediction (Figure 3.25).

To establish the critical length of steel section \( (L_s_{cr,a}) \) that would cause bond failure and CFFT failure to occur simultaneously, the moment calculated from Equation [3.3] and that calculated based on bond failure are set to be equal. One can then solve for \( (L_s/L)_{cr,a} \) by trial and error. Using this approach for the specimens tested in Phase II gave \( (L_s/L)_{cr,a} \) equal to 0.173 (shown in Figure 3.25). Detailed calculations are given in Appendix A.
3.3.4 Plastic Hinge Capacity

At large \((L_s/L)\) ratios, a plastic hinge is fully developed at the fixed end (section (a-a), Figure 3.29). This cross-section comprises the steel section, part of the concrete core in compression and part of the tube in compression. The tube is not effective in tension since it is a discontinuous end. A strain compatibility-equilibrium approach is adopted to establish the neutral axis depth and solve for the forces and ultimate moment \(M_u\) based on the following assumptions: the steel section is fully plastic and experiencing the ultimate strength level \(f_u\), concrete confinement is ignored in flexure given the steep strain gradient, the tube does not control failure in axial compression, and the equivalent stress block of 0.85\(f'_c\) stress extends over the entire depth of compression zone, \(c\) (this compensates for neglecting some concrete confinement). The predicted \(M_u\) using this method showed very good agreement for specimen CS4 but slightly underestimated the strength of CS5 (Figure 3.25).

To establish the critical length of steel section \((L_{x,cr,b})\) that would cause CFFT failure to occur simultaneously with the plastic moment capacity, the moment calculated from Equation \([3.3]\) and that calculated for the plastic moment are set equal to each other, and one can solve for \((L_s/L)_{cr,b}\). Using this approach for the specimens tested in Phase II gave \((L_s/L)_{cr,b}\) equal to 0.475 (shown in Figure 3.25). Detailed calculations are given in Appendix A.

3.3.5 Flexural Response of CFFT-Encased Steel Section

In this section, the program Response 2000 (Bentz, 2000), which is readily available for reinforced concrete analysis, was used to determine if this reinforced concrete analysis program could accurately predict the full response and failure load of the CFFT-encased steel S-sections. The assumptions made for material strengths and cross-sectional areas are also presented. Some of the primary sources of error are described.
3.3.5.1 Program Description

Response 2000 predicts the full response of reinforced concrete members under axial, flexural, and shear loads. It is a non-linear analysis program, which assumes that plane sections remain plane throughout loading. The overall member behaviour (moment-curvature) is predicted, as well as stresses and strains in the cross-section. The cross-sectional behaviour, in terms of stress and strain distributions, can be seen at many different points in the moment-curvature relationship.

3.3.5.2 Material Properties

The mechanical properties of the concrete, steel, and FRP tube that comprise the cross-section are described here. Also described here are the methods used to derive the properties of the FRP tube, since steel is the only reinforcing material available in the program and so certain properties had to be modified to represent the tube.

3.3.5.2.1 Concrete Core

The concrete in the Phase I specimens had an average strength of 37.1MPa. Default values were used for concrete tension strength, peak strain, and tension stiffening. Post-peak behaviour was assumed to be plastic due to partial confinement of the concrete (Mirmiran, 1998). Failure strains were also assumed to exceed 0.0035, as partial confinement would prevent concrete from crushing (Fam and Rizkalla, 2003).

3.3.5.2.2 Steel Sections

The steel section was modeled as discrete reinforcing bars whose behaviour is based on coupon tests. The steel stress-strain curve is made up of three portions: the initial linear-elastic region described by the modulus of elasticity and yield strength, the fully plastic yield portion,
and the strain hardening section starting at an inputted strain value and continuing to a maximum stress and strain values. All of these values were based on the coupon tests and are listed in Table 3.2.

3.3.5.2.3 FRP Tube

The only reinforcing material available in Response 2000 is steel, although the steel properties are customizable based on a few important points. The default stress-strain curve was modified to approximate the bilinear behaviour of the FRP coupons with a linear elastic-plastic curve. The initial linear portion of the curve represented the actual behaviour of the coupon, while the plastic line ensured that the analysis continued until the tube actually reached the failure strain and the full curve was stopped at the actual failure strain of the beam. The behaviour of the ±55° tubes could not be appropriately modeled, due to its significant non-linearity and the fact that the fibres never actually ruptured.

3.3.5.3 Model Construction

Each CFFT was represented by an area of concrete surrounded by reinforcing bars (Figure 3.30). The diameter of the concrete cylinder was identical to that of the actual specimen. The diameter of the longitudinal reinforcement used to represent the FRP tube was the same as the tube thickness, while the number of bars was determined based on the total cross-sectional area of the tube. The areas of FRP in the model and in the actual tubes are listed in Table 3.3.

Models were created for each piece of steel on its own to ensure that the model could accurately predict the behaviour of the S-section using distinct pieces of rebar. The flange of the steel was comprised of individual bars in a horizontal layer and the web was comprised of several bars evenly spaced vertically. Several diameters of steel reinforcing bars, chosen based on the taper of the flange, were used so that the model contained the same amount of steel as the beams.
The thickness of the web determined the diameter of the bars used in the model. Final areas of steel in the webs and flanges used in the models are listed in Table 3.4. It was necessary to include some concrete in the model for the bare steel sections, so a 10mm by 10mm square of concrete in tension is included in the model. The contribution of this amount of concrete in tension was negligible compared to the forces in the steel.

The behaviour of the CFFT-encased steel S-sections was modelled by combining the two models of the individual components described above, as shown in Figure 3.31.

3.3.5.4 Results

This section will present the results of the models described above and compare them to the corresponding experimental results. The ultimate loads are compared in Table 3.5. Comparison between Response 2000 results and experimental results of CFFTs and CFFT-encased steel S-sections, where a positive difference means that the Response 2000 model reached a higher load than the experimental value.

3.3.5.4.1 CFFTs

The Response 2000 model of the CFFTs, shown in Figure 3.32, showed good agreement with the experimental data in terms of both stiffness and ultimate moment and curvature. In the model, cracking of the concrete did not show the local decrease in strength and stiffness that was observed during testing. Better agreement was shown at service loads, above cracking but before ultimate. The Response 2000 curves were ended at a strain corresponding to the end of the linear-elastic part of the curve for the FRP. Variability at ultimate loads was in the order of 5-8%.
3.3.5.4.2 Steel beams

The steel beam models showed good agreement with the test data, as can be seen in Figure 3.33. The yielding of the steel in the model occurred at a lower load and was less of a sharp change than that observed during the steel beam tests. The approximation of a steel beam as discrete bars makes a difference in the behaviour of the steel of 3-7%.

3.3.5.4.3 CFFT-encased steel S-sections

The moment-curvature of the CFFT-encased steel S-sections generally agreed with the Response 2000 data shown in Figure 3.34. The ultimate moments and curvatures of the Response 2000 model were underestimated, most likely due to the inability of the discrete bars to account for the effect of the circumferential fibres at ultimate loads. Response 2000 also did not predict the post-peak response after first failure as the model as the FRP was assumed to behave plastically with a slow decrease in load.

3.3.5.5 Sources of Error

There were several sources of error in using a reinforced concrete analysis program to model CFFT-encased steel S-sections. In terms of the concrete material properties, the use of default values for tensile strength and tension stiffening and the assumption of post-peak plasticity of the concrete may not be entirely accurate as these were not quantified during the testing phase. These would be small sources of error, since the values used and assumptions made were based upon previous testing and typical behaviour of concrete. The linear elastic-plastic behaviour of the FRP is another source of error. As seen in the coupon test results in Figure 3.1, all of the coupons from the angle-ply tubes showed bilinear behaviour prior to rupture. This is typically a small underestimation of the stress in the FRP at a given strain level. This also means
that the behaviour of the model once plasticity of the FRP is reached is no longer valid and the post-rupture load capacity cannot be estimated by the model.

The model itself required approximations of the areas of steel and FRP as discrete bars. The area was as close as possible to the actual area of the experimental steel and FRP; however, some variability existed and the areas in the model were occasionally larger than the actual area. The steel was seen to have minor differences between the model and the experiments. The FRP behaviour near ultimate was affected more by the fact that it was modeled as discrete bars as there is no way to model the interaction between the layers of fibres which occurs in a continuous tube, or the effect of the circumferential fibres near ultimate load. This is likely the main factor that accounts for the underestimation of the strength of the CFFT and CFFT-encased steel S-sections at ultimate loads by 0.5-8%.
Table 3.1. Test matrix in Phases I and II

(a) Phase I: Beam flexural specimens

<table>
<thead>
<tr>
<th>Spec. ID</th>
<th>GFRP Tube ID</th>
<th>Tube Outer Diameter (mm)</th>
<th>Tube Thickness (mm)</th>
<th>Fibre Angles (°)</th>
<th>Steel I-section</th>
<th>Reinforcement index, ω</th>
<th>Span (mm)</th>
<th>Shear Span (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFFT1</td>
<td>T1</td>
<td>169</td>
<td>3.55</td>
<td>[-86/+9]</td>
<td>N/A</td>
<td>0.41</td>
<td>1180</td>
<td>420</td>
</tr>
<tr>
<td>CFFT1-S1</td>
<td>S1</td>
<td></td>
<td></td>
<td></td>
<td>N/A</td>
<td>1.13</td>
<td></td>
<td></td>
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<tr>
<td>CFFT2</td>
<td>T2</td>
<td>169</td>
<td>5.19</td>
<td>[-85/+6]</td>
<td>N/A</td>
<td>0.78</td>
<td>1180</td>
<td>420</td>
</tr>
<tr>
<td>CFFT2-S1</td>
<td>S1</td>
<td></td>
<td></td>
<td></td>
<td>N/A</td>
<td>1.66</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CFFT3</td>
<td>T3</td>
<td>114</td>
<td>4.01</td>
<td>[-84/+6]</td>
<td>N/A</td>
<td>0.81</td>
<td>800</td>
<td>285</td>
</tr>
<tr>
<td>CFFT3-S2</td>
<td>S2</td>
<td></td>
<td></td>
<td></td>
<td>S2</td>
<td>2.15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CFFT4</td>
<td>T4</td>
<td>115</td>
<td>5.07</td>
<td>[+54/-56]</td>
<td>N/A</td>
<td>N/A</td>
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<td>S2</td>
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<td></td>
<td></td>
<td>N/A</td>
<td>800</td>
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(b) Phase II: Cantilever specimens for connections

<table>
<thead>
<tr>
<th>Spec. ID</th>
<th>GFRP Tube ID</th>
<th>Outer Diameter D, (mm)</th>
<th>Tube Thickness (mm)</th>
<th>Fibre Angles (°)</th>
<th>Steel I-section</th>
<th>Embedded Steel Length L_s, (mm)</th>
<th>Span, L (mm) (exact)</th>
<th>(L_s/D_o)</th>
<th>(L_s/L)</th>
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<tr>
<td>CS1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>80</td>
<td>730 (745)</td>
<td>0.7</td>
<td>0.107</td>
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<td></td>
<td></td>
<td>140</td>
<td>730</td>
<td>1.2</td>
<td>0.192</td>
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<td>CS3</td>
<td>T3</td>
<td>114</td>
<td>4.01</td>
<td>[-84/+6]</td>
<td>S2</td>
<td>230</td>
<td>730</td>
<td>2.0</td>
<td>0.315</td>
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<td>CS4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>S2</td>
<td>400</td>
<td>730 (725)</td>
<td>3.5</td>
<td>0.552</td>
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<td>CS5</td>
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<td></td>
<td></td>
<td>800</td>
<td>730 (725)</td>
<td>6.3</td>
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Table 3.2. Material properties used in Response 2000 modeling

<table>
<thead>
<tr>
<th>Material</th>
<th>Modulus of Elasticity (GPa)</th>
<th>Yield Stress (MPa)</th>
<th>Onset of Strain Hardening (mm/m)</th>
<th>Maximum Strain (mm/m)</th>
<th>Maximum Stress (MPa)</th>
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<tr>
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<td>195</td>
<td>400</td>
<td>25</td>
<td>120</td>
<td>475</td>
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<tr>
<td>S2</td>
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<td>370</td>
<td>25</td>
<td>200</td>
<td>460</td>
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<tr>
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<tr>
<td>T3</td>
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<td>50</td>
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Table 3.3. Cross-sectional areas of models compared to cross-sectional areas of experimental samples for components in four-point bending tests of CFFTs and CFFT-encased steel S-sections

<table>
<thead>
<tr>
<th>Steel ID</th>
<th>Model Area (mm$^2$)</th>
<th>Sample Area (mm$^2$)</th>
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<tbody>
<tr>
<td>CFFT1</td>
<td>1898</td>
<td>1900</td>
</tr>
<tr>
<td>CFFT2</td>
<td>2700</td>
<td>2685</td>
</tr>
<tr>
<td>CFFT3</td>
<td>1365</td>
<td>1375</td>
</tr>
<tr>
<td>S1</td>
<td>1550</td>
<td>1545</td>
</tr>
<tr>
<td>S2</td>
<td>1179</td>
<td>1180</td>
</tr>
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</table>

Table 3.4. Comparison of web, flange, and total areas of the steel S-sections in the model to samples

<table>
<thead>
<tr>
<th>Steel ID</th>
<th>Area of Flange (mm$^2$)</th>
<th>Area of Web (mm$^2$)</th>
<th>Total Cross-sectional Area (mm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Model</td>
<td>Actual Beam</td>
<td>Model</td>
</tr>
<tr>
<td>S1</td>
<td>546</td>
<td>543.7</td>
<td>458</td>
</tr>
<tr>
<td>S2</td>
<td>363</td>
<td>363.2</td>
<td>453</td>
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Table 3.5. Comparison between Response 2000 results and experimental results of CFTTs and CFFT-encased steel S-sections.

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>Experimental Strength</th>
<th>Model Strength</th>
<th>Difference</th>
<th>%</th>
</tr>
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<tr>
<td></td>
<td>kNm</td>
<td>kNm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1: $M_y$</td>
<td>22.65</td>
<td>23.86</td>
<td>+5.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$M_u$</td>
<td>23.66</td>
<td>24.88</td>
<td>+5.2</td>
</tr>
<tr>
<td>S2: $M_y$</td>
<td>11.02</td>
<td>11.44</td>
<td>+3.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$M_u$</td>
<td>11.17</td>
<td>11.95</td>
<td>+7.0</td>
</tr>
<tr>
<td>CFFT1</td>
<td>22.52</td>
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<td>-5.7</td>
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</tr>
<tr>
<td>CFFT1-S1</td>
<td>49.66</td>
<td>47.60</td>
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<td>CFFT2</td>
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<td>CFFT2-S1</td>
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<td>-8.7</td>
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<tr>
<td>CFFT3</td>
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<td>CFFT3-S2</td>
<td>23.28</td>
<td>23.07</td>
<td>-0.9</td>
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</table>
Figure 3.1. Stress-strain curves of GFRP tubes used in test program

Figure 3.2. Stress-strain curves of steel beams used in test program
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Figure 3.6. Photo of test setup for Phase I – Beam flexural specimens

Figure 3.7. Schematic test setup for Phase II – Cantilever specimens for moment connections
Figure 3.8. Photo of test setup for Phase II – Cantilever specimens for moment connections

Figure 3.9. Effect of adding steel S-sections to CFFT members of different tube thicknesses on load-deflection response
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Figure 3.26. Steel-concrete bond failure mode in the analytical model
Figure 3.27. Bond-slip response (taken from Park and Paulay, 1975)

Figure 3.28. CFFT flexural failure method in the analytical model

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Chapter 4

Moment Connection Tests on Unreinforced CFFTs

This chapter examines the behaviour of CFFTs connected to concrete footings through a combination of direct embedment of the CFFT into the footing and steel dowels connecting the footing to the CFFT. The steel dowels extend into the CFFT within the CFFT portion embedded into the footing. Earlier research by Ahmad et al. (2008) on the shear strength of CFFT members without internal longitudinal steel reinforcement in a simple beam setup revealed that shear failure did not occur in short CFFT beams and that flexural strength was not compromised. This study addresses the effect of combined flexure and shear on the strength of the CFFT member in a cantilever setup, including the influence of the connection arrangement.

4.1 Experimental Program

4.1.1 Specimen Layout

Five CFFT specimens were fabricated and tested to determine the influence of combined flexural and shear loading on their strength as well as the capacity of the moment connection. Two of the CFFT specimens were tested as simply supported beams in bending (Specimens B1 and B2), whereas the other three specimens were tested as cantilevers (Specimens C1, C2, and C3). Details of the specimens are given in Table 4.1. All specimens were fabricated using tubes that had an outer diameter of 219mm. Material properties of the tube can be found in Section 4.1.2.1.

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2 This chapter has been published as the following ACI paper: Zakaib, S., Sadeghian, P., and Fam, A. (2010). “Influence of Connection Detailing on Strength of Concrete-Filled FRP Tubes Under Bending and Shear”. ACI Special Publication, SP 275, p. 1-16.
4.1.1.1 B1

In all previous research on CFFT members tested in flexure, three- or four-point bending tests were carried out where the CFFT member extends from the constant moment region into the shear span. Often, failure occurred just under one of the loading points and hence it is difficult to isolate the effects of flexure and shear. In this study, Specimen B1 was designed such that a CFFT segment is subjected to pure bending within a constant moment region, without the influence of shear, and is shown in Figure 4.1. The specimen was a simple beam loaded in four-point bending. The shear spans extending over the end supports consisted of heavily reinforced concrete blocks, 800 mm long, with a square cross-section of 400x400 mm. The CFFT tube was located within the constant moment region and embedded 300 mm on each side into the concrete blocks. The clear length of the CFFT member within the constant moment region was 660 mm or 3D, where D is the outer tube diameter. The span between supports was 1900 mm, with a shear span of 500 mm long on each side and a constant moment zone of 900 mm between the loading points, which ensures that the entire CFFT clear length is subject to pure bending. The connection between the tube and block was in the disturbed region near the loading point, which caused premature failure (which will be discussed further in Section 4.2.1). Specimen B2 was created and tested to determine the maximum moment relating to tensile rupture of the tube due to the premature failure of B1.

4.1.1.2 B2

Specimen B2 was loaded in three-point bending with unrestrained ends (i.e. no concrete blocks and slip between the tube and concrete core is unrestrained), as shown in Figure 4.2. The span length was 995 mm, giving a shear span of 497.5 mm, which is a shear-span-to-diameter ratio of 2.25. The objective is to examine the effect of a severe change in moment over the short
span on flexural strength of the specimen. In this specimen, given the relatively low shear-span-to-diameter ratio, a considerable arching action may develop.

4.1.1.3 C1

Cantilever specimen C1 was one of three CFFTs loaded with a single point load at the free end, and fixed at the other end. The span of specimen C1 was 1040 mm, or 4.75D. The fixed end consisted of a tube encased in a reinforced concrete footing with all sides equal to 500 mm. The tube was embedded 260 mm into the concrete block and connected by steel dowels within the embedment length as will be described. To create a flat loading point at the free end, the CFFT was also encased in a concrete block of equal side lengths of 400 mm in each direction, but was embedded only 220 mm due to the very low moment at the free end. Figure 4.3 shows a typical setup used for specimens C1 and C2. Specimen C1 is intended to study the connection under the combined shear and bending for a shear span-to-diameter ratio of 4.75.

4.1.1.4 C2

Specimen C2 was also loaded as a cantilever embedded in a concrete block for fixity, using a similar joint to that in specimen C1. The main difference between specimens C2 and C1 is the span length which was 1600 mm or 7.25D for C2. The test setup is similar to that shown in Figure 4.3.

4.1.1.5 C3

Specimen C3 had the same span as specimen C1, 1040 mm or 4.75D, and was also a cantilever specimen. The difference is that fixed end for this specimen is the result of mechanical clamping of the tube between two plaster-filled steel channel sections. The tube was clamped
over a length of 610 mm and loaded directly at the other end where a short overhang of 482 mm was provided to allow for rotation of the beam. Figure 4.4 shows a schematic of the test setup.

The objective of testing specimen C3 was to see the effect on the moment capacity of a close-to-perfect fixed end where no slip would occur, and compare this behaviour to an equal span specimen C1 that has a practical moment connection to a concrete footing. Also, specimen C3 is compared to a specimen that was tested by Mitchell and Fam (2010), with the same tube and fixed end arrangement, but with a 2665 mm or 12.2D span, to examine the effect of shear span-to-diameter ratio.

4.1.2 Material Properties

4.1.2.1 FRP Tubes

The tubes used for all tests were identical. The E-glass and epoxy tubes were fabricated for transmission of fluids. They have an outer diameter of 219 mm and wall thickness of 4.3 mm, in a 7-ply [-86/+6/-86/+6/-86/+6/-86] lay-up alternating layers of hoop and longitudinal fibres. The angles of each layer (in degrees) are described with respect to the longitudinal axis of the tube. Tensile tests were performed on coupons cut from the tube in the longitudinal direction. The coupons had a width of 25 mm, tabs that are 115 mm long at both ends, and 37 mm gauge length between grips. The small gauge length was chosen to decrease the effect of the discontinuous fibres wound at 6° from the longitudinal axis of the tube, following the recommendation by Mandal (2004) that a shorter coupon will result in closer behaviour and failure strain to that of the full tube. Electric resistance strain gauges were placed on either side of the coupon at the centre of the gauge length. Figure 4.5 shows the stress-strain curve of the GFRP tube in tension, which is somewhat bilinear, with an initial modulus of 19.5 GPa. The ultimate strength and strain were
250 MPa and 0.0205, respectively. The modulus changed to 10.7 GPa at a stress of 43 MPa and a strain of 0.0045.

4.1.2.2 Concrete

Table 4.1 shows the concrete compressive strength based on standard concrete cylinder tests at the time of the test. The concrete strength varied from 30 to 41 MPa. A previous study by Fam and Rizkalla (2003) showed that the influence of concrete strength on flexural strength of CFFT members is insignificant, which is quite different from the case of axially loaded CFFT members in compression, due to the confinement effect. For the concrete blocks, the compressive strength at the time of testing ranged from 29 to 31 MPa.

4.1.2.3 Steel

Grade 400W steel reinforcing bars were used in specimens B1, C1, and C2. Material testing was not undertaken for the steel as it was not part of the system being tested and failure did not depend on the material properties of the reinforcing bars. A combination of 10M and 15M bars were used for longitudinal and transverse reinforcement in the concrete blocks. Four 20M dowels, bent at 90°, were used to connect each tube to the concrete blocks. The dowels were extended into the tube a distance equal to the embedment length of the tubes in the concrete blocks.

4.1.3 Fabrication

Specimens B1, C1, and C2 were all fabricated at the same time, in two separate stages. The first stage involved filling the tubes whilst in an upright position and inserting the dowels into each end. The tubes were then laid horizontally into the formwork of the end blocks and the concrete was poured at a later stage. This practice was carried out for convenience in the lab, but
for practical applications, the footing will be cast before filling the tube. Figure 4.6 shows the typical fabrication process for specimen B1. Specimens B2 and C3 were simply filled with concrete in an upright position.

### 4.1.4 Test Set-up

Figure 4.1-4.4 show the test setups of all specimens. In testing specimen B1 as a simple beam, the loading and supporting points were all relative to the two rigid concrete prisms, ensuring pure bending over the CFFT segment. A hinged support was provided at one end while a roller support was provided at the other end. Specimen B2 was also simply supported, using a hinge and roller at either end. A plaster-filled short steel channel section was used at each support and at the loading point to conform to the round surface of the CFFT of specimen B2. For cantilever specimens C1 and C2, the concrete blocks at the fixed ends were clamped using a heavy HSS steel section, anchored to the machine using 25 mm diameter high-strength threaded rods. Specimen C3 was also clamped using HSS sections anchoring the steel channel sections to the machine. Figure 4.7 shows pictures of the test setups of all specimens. A 900 kN capacity mechanical testing machine was used to test all specimens under stroke control.

### 4.1.5 Instrumentation

Strain measurements were taken using electrical resistance strain gauges and position indicator (PI) gauges. Deflection and slip measurements were taken using linear potentiometers (LPs). The locations of all instrumentation can be seen for each test in Figure 4.1-4.4. Load was measured through a load cell built in to the machine.
4.2 Test Results

Figure 4.8 shows the load-deflection responses of all test specimens, while Figure 4.9 shows the load-longitudinal strain responses at the maximum moment regions. In the following sections, failure modes and the behaviour are discussed in detail.

4.2.1 Failure Modes

Specimen B1 reached a load level very close to that corresponding to flexural failure of the CFFT member, as evidenced by the tensile strains in Figure 4.9; however, slip occurred between the CFFT member and the concrete block portion, leading to a significant load drop. Slip was associated with radial cracking of the concrete block as shown in Figure 4.10a). However, as indicated from the tensile strains, flexural tension failure was imminent. The tensile strain in the tube at failure was 0.0199, while the average tensile failure strain of the tube from the coupon tests is 0.0205. Specimen B2 failed in tension at mid-span by rupture of the GFRP tube, as shown in Figure 4.10b). The strain at failure was 0.0243. Some slip occurred between the concrete core and the tube, as shown in Figure 4.11a).

All three cantilever specimens (C1, C2, and C3) failed through tensile rupture of the tube, as shown in Figure 4.10b). Figure 4.12 shows the load-slip responses of cantilever specimens C1 and C2. The slip is measured between the GFRP tube and the concrete footing; essentially the tube is pulled out of the concrete block. It is clear that slip in specimen C2 was significantly larger than that of C1. Clearly, there appears to be a correlation between the span length and the amount of slip, as the fixed end arrangements were identical in both specimens. Specimen C2 slipped significantly more than specimen C1, as shown in Figure 4.11b), and both footings had some radial cracks around the tube.
Specimen C3, which was not embedded in a concrete footing but rather clamped mechanically, failed also by rupture in tension at the highest moment capacity of all three cantilever specimens, though very close to that of C1 and B1. No slip was observed in specimen C3, and it failed at a tensile strain of 0.0240.

4.2.2 Moment-Curvature Responses

Since the boundary conditions and spans varied among the test specimens, the behaviour of the different specimens was compared based on the moment-curvature responses. The moment-curvature behaviour of the five specimens is shown in Figure 4.13. The curvature was determined as the slope of the strain profile at the maximum moment locations. Generally, all specimens showed similar behaviour, with the first cracking of concrete occurring around a moment of 10 kNm. This is followed by a reduction in stiffness and slightly nonlinear behaviour. The nonlinear behaviour results from the combined effects of concrete nonlinearity, increased cracking and the tube bi-linear stress-strain behaviour associated with splitting of the hoop fibres. Over the entire set of tests, the maximum moment achieved was 59.4 kNm for Specimen B2 and the average moment was 53.4 kNm, not including specimen C2. Specimen C2 failed at a significantly lower moment of 31.5 kNm, showing a 40% reduction in capacity relative to the other specimens.

It is worth noting that identical CFFT using the same tube used in this study and a concrete fill of 44 MPa compressive strength was tested by Fam and Mitchell (2010), using a setup similar to that of specimen C3 of this study but with a span of 2.665 m, which is equivalent to 12.2D. That specimen had flexural tension failure by rupture of the tube at a moment of 56.9 kNm. The shear span of this specimen is long enough to assure that shear did not affect the flexural strength of the specimen. Also, the moment capacity of this specimen is quite consistent
with the value obtained in this study for specimen B1, based on a slight extrapolation of the load-strain response to hypothetically reach the rupture strain of the tube, which although was imminent, was not reached because of the slip failure. In the same study by Fam and Mitchell (2010), a hollow tube was also tested under the same conditions and achieved 29.63 kNm, giving the lower bound strength of the system. This information will shed light on the behaviour of the specimens tested in this study, as will be discussed in the following sections.

4.2.3 Effect of Shear Span Length of Cantilevered CFFTs

The shear span lengths were determined so that there would be enough of a difference between the short cantilevers (C1 and C3), specimen C2, and the specimen tested by Fam and Mitchell (2010). These were 4.75D, 7.25D, and 12.2D, respectively. The short cantilevers had to have a shear-span-to-depth ratio much greater than 1, which was found to be the transition point between shear and flexural failure modes (Ahmad, 2004). Material availability and constraints based on the loading setup also factored into the choice of shear spans.

4.2.3.1 Mechanically clamped end

Specimens C3 and the one tested by Fam and Mitchell (2010) were similar in test setup and fixed end arrangements. The main difference was the span lengths, which were 4.75D and 12.2D, respectively. It is noted from Figure 4.13 that the moment capacity of specimen C3 is only 7% lower than the other one, despite the significantly shorter span and hence the higher influence of shear. Given the variability of concrete fill strengths and the well-established possibilities of variation of rupture strains of the tube, this 7% reduction is not significant enough to be attributed to the influence of higher shear forces. As such, it may be concluded that shear does not lead to reduction in flexural strength in fixed-end cantilevered CFFT members. This confirms the findings established earlier for CFFT simply supported beams by Ahmad et al. (2008).
4.2.3.2 Concrete footing with fixed end

Specimens C1 and C2 both had the same fixed-end arrangements and detailing, using a concrete footing. They had shear spans of 4.75D and 7.25D, respectively. While Specimen C1 with the shorter span achieved very close to, but not quite, its potential flexural strength and failed at a moment of 49.2 kNm, Specimen C2, with the longer span and therefore lower shear, failed at a significantly lower moment of 30.2 kNm. It is important to note that both specimens failed by tensile rupture of the tube. It is also worth noting that the slip in Specimen C2 at failure was 11 mm, which is significantly larger than that in C1 at the same load level, 1.3 mm, as shown in Figure 4.12. The slip in Specimen C1 at failure was only 3 mm.

This behaviour is very significant; it clearly points out that achieving tension failure (rupture) of the tube does not guarantee achieving the full potential flexural moment capacity of the CFFT member. The reduction in moment capacity is clearly attributed to the influence of slip of the concrete core, at the critical maximum moment cross-section. The concrete core inside the tube, within the segment embedded in the footing was well anchored to the footing through the steel dowel bars. As such, the observed CFFT slip from the footing (Figure 4.11b) was in fact a slip of the GFRP tube, relative to both the external and internal concrete (i.e. the footing and the concrete core segment within the footing, respectively). This behaviour is illustrated in Figure 4.14a). The result is widening of the internal flexural crack in the concrete core inside the tube at the footing face, due to that slip. This leads to significant reduction in the size of the intact concrete core compression zone, or even a complete separation in the entire concrete core at large slip, meaning a complete loss of the concrete compression zone. The cross-section at this point could be somewhat similar to that of a hollow GFRP tube. This is supported by the observed very low moment capacity of specimen C2, which is in fact very close to that of the hollow tube tested.
by Fam and Mitchell (2010). This problem could be avoided either by embedding the CFFT an additional length inside the footing to reduce slip, or extending the steel dowels further into the CFFT.

It is important to note that a similar behaviour also occurs in short simply supported CFFT beams, where the unrestrained slip at the ends (Figure 4.11a) leads to a reduction of the concrete core compression zone size, as shown in Figure 4.14b), and hence a reduction in moment capacity. However, because of the concrete arching action occurring in short and deep beams, the strength enhancement due to the arching action compensates for the weakening effect arising from the reduction of the compression zone size. The apparent result is no loss of the moment capacity or even a little gain as was observed in specimen B2 (based on low shear span and arching action), and also reported earlier by Ahmad et al. (2008).

4.2.4 Effect of Fixed End Arrangement

Specimens C1 and C3 both had the same shear span of 4.75D but C1 had a moment connection through a concrete footing detailing, while C3 was mechanically clamped. In this case both specimens achieved a comparable moment capacity, with specimen C1 only 3.5% lower than C3, as shown in Figure 4.12. This suggests that the moment connection succeeded for this particular shear span length. However, clearly the premature failure of specimen C2 with the same moment connection but longer shear span suggests that the connection is vulnerable to significant slippage and cracking as the span gets longer.
Table 4.1. Test matrix

<table>
<thead>
<tr>
<th>Spec. ID</th>
<th>Loading Type</th>
<th>Span (mm)</th>
<th>Shear Span (x D)</th>
<th>Embedment (mm)</th>
<th>$f'_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>4-point bending</td>
<td>1900</td>
<td>N/A</td>
<td>300</td>
<td>41</td>
</tr>
<tr>
<td>B2</td>
<td>3-point bending</td>
<td>1000</td>
<td>2.25</td>
<td>N/A</td>
<td>30</td>
</tr>
<tr>
<td>C1</td>
<td>Cantilever</td>
<td>1040</td>
<td>4.75</td>
<td>260</td>
<td>36</td>
</tr>
<tr>
<td>C2</td>
<td>Cantilever</td>
<td>1600</td>
<td>7.25</td>
<td>260</td>
<td>34</td>
</tr>
<tr>
<td>C3</td>
<td>Cantilever</td>
<td>1040</td>
<td>4.75</td>
<td>(clamped) 610</td>
<td>30</td>
</tr>
</tbody>
</table>
**Figure 4.1.** Test setup of specimen B1 (all dimensions in mm)

**Figure 4.2.** Test setup of specimen B2 (all dimensions in mm)
Figure 4.3. Test setup of specimens C1 and C2 (all dimensions in mm)

Figure 4.4. Test setups of specimen C3 and specimen tested by Mitchell and Fam (2010) (all dimensions in mm)
Figure 4.5. Stress-strain curve of tubes from coupon testing

(a) CFFT with end dowels, b) reinforcement of end blocks, and c) casting the end block
Figure 4.7. Test setups for specimens: (a) B1, (b) B2, (c) C1 and C2 and (d) C3
Figure 4.8. Load-deflection behaviour of specimens

Figure 4.9. Load-strain behaviour of test specimens
Figure 4.10. Failure modes of (a) Specimen B1 and (b) Specimens B2 and C1 to C3

Figure 4.11. (a) Internal CFFT concrete slip, and (b) External CFFT concrete slip
Figure 4.12. Load-slip responses of C1 and C2

Figure 4.13. Moment-curvature behaviour of test specimens
Figure 4.14. Influence of concrete core slip on compression zone size
Chapter 5

Conclusions and Future Research

5.1 Flexural Behaviour of CFFT-encased Steel S-sections

Based on the experimental and analytical investigations, the following conclusions can be drawn:

1. Providing a steel I-section encased in CFFTs with cross-ply [-85/+6] laminates results in increasing flexural strength, stiffness and ductility considerably. Upon flexural failure, by tensile rupture of the tube, the load drops to about two thirds of the peak strength and remains sustained over a large range of deflection as the steel section continues to yield.

2. CFFTs with angle-ply [±55] laminates are inherently quite ductile on their own, compared to CFFTs with cross-ply [-85/+6] laminates. They demonstrate a flexural response quite similar to the yield plateau in metals. As such, providing a steel I-section in these CFFTs enhanced considerably flexural strength and stiffness only, but not ductility.

3. A successful moment connection has been developed for CFFT members, where the embedded steel I-section is welded to a steel base plate, before casting the CFFT member, which sits directly on the plate. The strength and ductility of this connection depends primarily on the extent of embedment of the steel section into the CFFT member.

4. The minimum embedment length of the steel section, measured from the base plate, and required for the CFFT member to reach its flexural strength, was 17% of the CFFT span.
Failure in this case occurs at the free end of the steel section. Shorter embedment results in premature bond failure of the steel section. This was successfully modeled analytically.

5. The minimum embedment length required to reach the full plastic capacity of the moment connection at the fixed end is 48% of the CFFT span. This was also successfully modeled analytically.

6. The full plastic capacity of the moment connection is only about 80% of the flexural strength of the hybrid CFFT-steel member, since the tube is cut-off at the end section.

5.2 Connection Tests on Unreinforced CFFTs

In this study cantilevered CFFT members with different fixed end arrangements and different shear spans were tested. Additional CFFT control specimens were also tested as simply supported beams. The moment connections studied were a mechanically clamped fixed end and a CFFT embedment into a concrete footing with steel dowels extending from the footing into the concrete core of the CFFT, within the embedment length. The objectives of the study were to examine the effects of shear span length and moment connection detailing on flexural strength of CFFT cantilevered members. The following conclusions are drawn:

1. In short CFFT cantilevered members, the high level of shear does not cause shear failure or reduction in the ultimate moment capacity. A similar observation was reported in literature for simply supported CFFT beams.

2. Rupture of the GFRP tube in tension in the CFFT system is not necessarily an indication of achieving its full potential flexural strength. Slip between the concrete core and the tube in the vicinity of the moment connection could significantly reduce moment capacity, but with the tube still failing in tension.
3. The effectiveness of the CFFT-footing moment connection studied reduced as the span of the CFFT member increased, due to the increased slip. The connection was successful in developing the full CFFT flexural strength for the span of 4.75D but not for the case of 7.25D, where only 40% of the flexural strength was achieved. This observation needs further research to clarify.

It is recommended that concrete footings of larger dimensions than those used in this study be used, to avoid cracking of the footing. It is also recommended that the CFFT embedment into the footing be increased or the steel dowels embedment into the concrete core of the CFFT member increase, to reduce the possibility of slip which could lead to reduction in moment capacity.

5.3 Further research

Further research should be carried out to support conclusions reached in all three phases of testing and widen the range of applications for which they are valid. For the CFFT-encased steel S-sections, a wider variety of tube diameter, fibre architecture and thickness and steel reinforcement ratios should be tested to establish a design procedure for a wide range of beams. Field scale testing should also be done to verify conclusions made here.

To continue the connection tests for CFFT-encased steel S-sections, combined axial load and flexural loads should be studied to determine an interaction diagram for the system. Increases in capacity might also come from adhesives applied to the steel surface before casting or surface preparation to increase bond. Other connections could be tested, and some possibilities include: direct embedment into concrete, with steel embedment likely to be most effective; or steel plates bolted to the W-section. Pinned connections may also be tested to increase the efficiency of the construction process through prefabrication of CFFT-encased steel S-sections.
To attempt to increase the capacity of the unreinforced CFFT's, tests could be undertaken with the steel bars extending partially into the CFFT member (i.e. a longer distance than that used in this study) to determine the minimum length required to prevent slip failure.
References


REFERENCES


REFERENCES


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Appendix A

Sample Calculations for Analytical Model

Dimensions (Figure A1):

\[
D = 114 \text{ mm} \\
h = 76 \text{ mm} \\
b = 62.5 \text{ mm} \\
t_w = 5.97 \text{ mm} \\
t_t = 5.4 \text{ mm} \\
t_{FRP} = 4.01 \text{ mm} \\
f'_c = 35 \text{ MPa} \\
f_u = 450 \text{ MPa}
\]

**Ultimate CFFT Strength:**

From Fam and Son (2008):

\[
M_{u,CFFT} = 0.0045D_0^3f'_c\left(100\frac{4t}{D_0}f_{tu}\right)^{0.815}
\]

\[
= 0.0045(114^3)(35)\left(100\frac{4 \times 4.01}{114} \frac{202}{35}\right)^{0.815}
\]

\[
= 8.4 \text{ kNm}
\]

From Figure A.3.:

\[
M_{fixed, CFFT3} = 8.4 \left[\frac{L}{(L-L_s)}\right] \text{ kNm}
\]

\[
= 8.4 \times (730/500)
\]

\[
= 12.26 \text{ kNm}
\]

**Plastic Moment Capacity:**

Assume:

- Fully plastic steel throughout the cross-section
- Unconfined concrete, since it is in flexure
- Ignore tube in compression
- Concrete stress of 0.85f_c', using the ACI uniform stress block
- Concrete stress block extends to neutral axis (not \( \beta_c \)) to compensate for previous assumptions and for simplicity

Assuming that the neutral axis depth is in the web (Figure A.2):
\[ C_c = 0.85 f_c' A_c \]
\[ = 0.85 * 35 * \frac{114^2}{8} \left[ 2 \cos^{-1} \left( \frac{114/2 - c}{114/2} \right) - \sin \left( 2 \cos^{-1} \left( \frac{114/2 - c}{114/2} \right) \right) \right] \]
\[ = 48328.88 \left[ 2 \cos^{-1} \left( \frac{114/2 - c}{114/2} \right) - \sin \left( 2 \cos^{-1} \left( \frac{114/2 - c}{114/2} \right) \right) \right] \]

\[ C_{s1} = A_f f_u \]
\[ = 62.5 * 5.4 * 450 \]
\[ = 151875 \text{ N} \]

\[ C_{s2} = \left( c - t_{FRP} - \frac{D - 2t_{FRP} - h}{2} - t_f \right) t_w t_{fu} \]
\[ = \left( c - 4.01 - \frac{114 - 2 * 4.01 - 76}{2} - 5.4 \right) 5.97 * 450 \]
\[ = 2686.5 c - 65550.6 \]

\[ T_{s1} = A_f f_u \]
\[ = 62.5 * 5.4 * 450 \]
\[ = 151875 \text{ N} \]

\[ T_{s2} = \left( D - c - t_{FRP} - \frac{D - h - 2 * t_{FRP}}{2} - t_f \right) t_w f_u \]
\[ = \left( 114 - c - 4.01 - \frac{114 - 76 - 2 * 4.01}{2} - 5.4 \right) 5.97 * 450 \]
\[ = 240710.4 - 2686.5 c \]

\[ \Sigma C = \Sigma T \]
c = 39.60 mm

\[ C_c = 93.76 \text{ kN} \]
\[ C_{s1} = 151.88 \text{ kN} \]
\[ C_{s2} = 40.85 \text{ kN} \]
\[ T_{s1} = 151.88 \text{ kN} \]
\[ T_{s2} = 134.61 \text{ kN} \]

Distances from the neutral axis to the centroid of each force:

\[ y_c = \frac{4R \sin^3(\frac{\theta}{2})}{3(\theta - \sin \theta)} - \left(\frac{D}{2} - c\right) = 16.43 \text{ mm} \]
\[ y_{C,S1} = c - t_{FRP} - y - t_f/2 = 17.9 \text{ mm} \]
\[ y_{C,S2} = (c - t_{FRP} - y - t_f)/2 = 7.6 \text{ mm} \]
\[ y_{T,S1} = D - c - t_{FRP} - y - t_f/2 = 52.7 \text{ mm} \]
\[ y_{T,S2} = (h - c + t_{FRP} + y)/2 = 25 \text{ mm} \]

Taking moments about the neutral axis:

\[ M_r = C_c y_c + C_{s1} y_{C,S1} + C_{s2} y_{C,S2} + T_{s1} y_{T,S1} + T_{s2} y_{T,S2} \]
\[ = 15.95 \text{ kNm} \]

**Slip Failure:**

\[ \tau_u = 0.088 \hat{f}_{\text{cube}} \]
\[ \hat{f}_{\text{cube}} = 1.25 \hat{f}_{\text{cylinder}} \]

\[ \tau_u = 0.088 \times 1.25 \times 35 \text{ MPa} = 3.85 \text{ MPa} \]

**CS1:**

Perimeter of the steel section:

\[ P_1 = b + 2t_f + (b-t_w) \]
\[ = 62.5 + 2 \times 5.4 + (62.5 - 5.97) \]

A3
\[ P_2 = 2 \times (h - 2t) \]
\[ = 2 \times (76 - 2 \times 5.4) \]
\[ = 130.40 \text{ mm} \]

\[ P_3 = P_1 = 129.83 \text{ mm} \]

Distances to the centroids of the steel sections from the midline of the tube thickness:

\[ y_1 = D - 1.5t_{FRP} - g - t_f/2 \]
\[ = 114 - 1.5 \times 4.01 - 14.99 - 5.4/2 \]
\[ = 90.3 \text{ mm} \]

\[ y_2 = D/2 - t_{FRP}/2 \]
\[ = 114/2 - 4.01/2 \]
\[ = 55 \text{ mm} \]

\[ y_3 = t_{FRP}/2 + g + t_f/2 \]
\[ = 4.01/2 + 14.99 + 5.4/2 \]
\[ = 19.7 \text{ mm} \]

\[ L_s = 80 \text{ mm} \]

\[ M_{bond, CS1} = 3.85 \text{MPa} \times 80 \text{mm} \times [129.83 \text{mm} \times 90.3 \text{mm} + 130.40 \text{mm} \times 55 \text{mm} + 129.83 \text{mm} \times 19.7 \text{mm}] \]
\[ = 6.61 \text{kNm} \]

CS2:

\[ M_{bond, CS2} = M_{bond, CS1} \times \frac{L_s, CS2}{L_s, CS1} \]
\[ = 6.61 \text{kNm} \times \frac{140 \text{ mm}}{80 \text{ mm}} \]
\[ = 11.6 \text{kNm} \]
CS3:

\[ M_{\text{bond, CS3}} = M_{\text{bond, CS1}} \left( \frac{L_s, \text{CS3}}{L_s, \text{CS1}} \right) \]

\[ = 6.61 \text{ kNm (230 mm / 80 mm)} \]

\[ = 19.0 \text{ kNm} \]

BUT \( M_{\text{bond, CS3}} > M_{\text{CFFT, ult}} \)

**Transition Points:**

To find the critical lengths of steel that are the transition points between the failure modes, find the length where the ultimate strength of the CFFT is equal to the plastic moment at the fixed end.

\[ M_{\text{fixed end}} = M_{\text{plast}} \]

\[ 8.4 \left[ \frac{L}{(L - L_s)} \right] = 15.93 \]

\[ (L/L_s)_{\text{critical}} = 0.475 \]

To find the transition point between bond and ultimate CFFT strength, set the moments equal to each other.

\[ M_{\text{bond, crit}} = M_{\text{bond, CS1}} \left( \frac{L_s, \text{crit}}{L_s, \text{CS1}} \right) \]

\[ = 6.61 \text{ kNm (L_s, crit / 80mm)} \]

\[ = 0.0826 \left( \frac{L_s, \text{crit}}{80\text{mm}} \right) \]

\[ 8.4 \left[ \frac{L}{(L - L_s, \text{crit})} \right] = 82.63 \left( \frac{L_s, \text{crit}}{80\text{mm}} \right) \]

For \( L=730\text{mm}, L_s, \text{crit} = 125\text{mm} \)

\[ (L_s/L)_{\text{crit}} = 0.173 \]
Table A.1. Summary of analytical model capacities for the three potential failure modes

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Plastic Hinge Capacity (kNm)</th>
<th>Ultimate CFFT Capacity (kNm)</th>
<th>Bond Capacity (kNm)</th>
<th>Predicted Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS1</td>
<td>15.95</td>
<td>9.43</td>
<td>6.61</td>
<td>Bond</td>
</tr>
<tr>
<td>CS2</td>
<td>15.95</td>
<td>10.39</td>
<td>11.56</td>
<td>CFFT</td>
</tr>
<tr>
<td>CS3</td>
<td>15.95</td>
<td>12.26</td>
<td>18.99</td>
<td>CFFT</td>
</tr>
<tr>
<td>CS4</td>
<td>15.95</td>
<td>18.74</td>
<td>33.04</td>
<td>Plastic Hinge</td>
</tr>
<tr>
<td>CS5</td>
<td>15.95</td>
<td>12.18</td>
<td>59.88</td>
<td>Plastic Hinge</td>
</tr>
</tbody>
</table>

Figure A.1. Dimensions of the CFFT-encased steel I-sections
Figure A.2. Force distribution for plastic hinge failure model

Figure A.3. Bending moment diagram for moment connection of CFFT-encased steel I-sections