Behaviour of CFRP-Confined Hollow Core Reactive Powder Concrete Columns

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School of Civil, Mining and Environmental Engineering

Behaviour of CFRP-Confined Hollow Core Reactive Powder Concrete Columns

A thesis submitted in fulfilment of the requirements for the award of the degree of

DOCTOR OF PHILOSOPHY

By

Hussamaldeen Goaiz

May 2018
I, Hussamaldeen Goaiz, hereby declare that this thesis, submitted in fulfilment of the requirements for the award of Doctor of Philosophy, in the School of Civil, Mining and Environmental Engineering, University of Wollongong, is wholly my own work unless otherwise referenced or acknowledged. The document has not been submitted for qualification at any other academic institution.

Hussamaldeen Goaiz
May 2018
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Finally, I would like to thank my wife Bayda and my sons Yousif and Abdulla for their love and continuous support and motivation during all stages of my PhD study.
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Journal papers


Conference papers


ABSTRACT

Hollow core concrete columns have been widely used when low weight and low cost design is required as a result of reducing the amount of concrete in the structural members. Also, using Fibre Reinforced Polymer (FRP) materials in concrete retrofitting or new concrete constructions is preferred to steel because of its lower self-weight and higher corrosion resistance than steel. Existing studies have shown that the use of an internal tube can significantly enhance the effectiveness of confinement in FRP-confined hollow columns. The internal tube used in the existing studies, however, generally had a large stiffness and also served as longitudinal reinforcement, such as steel or FRP tubes. The use of a stiff internal tube is inefficient in resisting bending for hollow columns with a relatively small void, and may be unnecessary for constraining the inner surface of concrete. Against this background, this study presents a new type of FRP-confined hollow columns with an internal PVC tube. In such cases, the main function of the internal PVC tube is to restrain the inner surface of concrete for effective confinement. The permanent internal PVC tube has also many other advantages in construction industry such as; low cost, excellent durability, long life expectancy, ease of fabrication and handling. The main disadvantage is the low fire resistance of the PVC material, however when the PVC is used as an internal tube in a hollow concrete section, it would be protected by a thick layer of concrete.

In order to investigate the behaviour of CFRP-confined hollow core concrete specimens with an internal PVC tube, 18 specimens were tested under concentric axial compression. The specimens had an outer diameter of 150 mm and a height of 300 mm. These specimens were divided into three groups according to the section configuration. The first group had an internal PVC tube; the second group were hollow cylinders with an inner void of 90 mm, while the third group were solid cylinders. The test variables
included the section configuration (i.e. solid specimens, hollow specimens and hollow specimens with a PVC tube) and the thickness of FRP. The test results showed that due to the beneficial effect of the PVC tube which provided constraints/confinement from inside, FRP-confined hollow columns with an internal PVC tube generally possessed good strength and ductility compared to their counterparts without a PVC tube.

In hollow core concrete columns, using concrete with ultra-high strength and ductility such as Reactive Powder Concrete (RPC) instead of normal strength concrete can be a preferable option for structural designers to compensate the reduction of the axial load capacity in concrete columns due to the effect of the hollow core. The combination of using an external FRP confinement, a hollow core RPC section and an internal PVC tube could result in a light-weight structural member with very high strength and ductility characteristics. In order to investigate the behaviour of CFRP-confined Hollow Core Reactive Powder Concrete (HCRPC) columns, 16 circular hollow core specimens (206 mm in diameter, 800 mm in height and a 90 mm circular hole) were made with RPC of 105 MPa compressive strength. These specimens were divided into four groups. The first group consisted of four unconfined specimens reinforced with longitudinal steel bars and steel helices. The specimens of the second group had the same configuration as the first group except that these specimens were externally confined with a CFRP tube. The specimens of the third group were externally confined with a CFRP tube and internally confined with a PVC tube. Finally, the specimens of the fourth group had no steel reinforcement and were only made with an external CFRP tube and an internal steel tube. These specimens were subjected to different loading conditions: concentric, eccentric (with eccentricities of 25 mm and 50 mm) and four-point bending. It was found that by introducing the PVC tube as internal confinement to the hollow columns both the strength and the ductility were improved compared to
those without internal PVC tube or with internal steel tube, especially under the loading conditions of concentric and four-point bending.

An analytical program (layer-by-layer numerical integration approach) was adopted to create axial load-bending moment ($P-M$) interaction diagrams for the CFRP-confined HCRPC specimens. According to the analytical results, the $P-M$ interaction diagrams of the HCRPC specimens can be modelled with an acceptable accuracy by using existing stress-strain models of both unconfined and CFRP-confined concrete.

Finally, the experimental and analytical results showed that the use of an internal PVC tube can enhance the performance of CFRP-confined HCRPC specimens in terms of strength and ductility compared to their counterparts without a PVC tube.
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<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>Area of single layer in layer-by-layer numerical integration approach</td>
</tr>
<tr>
<td>$b$</td>
<td>Width of single layer in layer-by-layer numerical integration approach</td>
</tr>
<tr>
<td>$cc$</td>
<td>Clear concrete cover</td>
</tr>
<tr>
<td>$d_{centre}(L)$</td>
<td>The distance from the centre of the layer to the centreline of the section</td>
</tr>
<tr>
<td>$d_h$</td>
<td>Nominal diameter of transverse steel helix reinforcement</td>
</tr>
<tr>
<td>$d_N$</td>
<td>Depth of the neutral axis</td>
</tr>
<tr>
<td>$d_{top}(L)$</td>
<td>The distance from the centre of the layer to the top of the section</td>
</tr>
<tr>
<td>$d_s$</td>
<td>Nominal diameter of steel bar reinforcement</td>
</tr>
<tr>
<td>$D_i$</td>
<td>Hollow core diameter of the concrete section</td>
</tr>
<tr>
<td>$D_o$</td>
<td>Outer diameter of the concrete section</td>
</tr>
<tr>
<td>$E_f$</td>
<td>Modulus of elasticity of FRP material</td>
</tr>
<tr>
<td>$E_s$</td>
<td>Modulus of elasticity of the steel bars reinforcement</td>
</tr>
<tr>
<td>$E_t$</td>
<td>Modulus of elasticity of the internal tube (PVC or steel)</td>
</tr>
<tr>
<td>$F(L)$</td>
<td>The total force response of the layers</td>
</tr>
<tr>
<td>$f'_c$</td>
<td>Cylinder concrete compressive strength at 28 days</td>
</tr>
<tr>
<td>$f'_{cc}$</td>
<td>Confined concrete strength</td>
</tr>
<tr>
<td>$f'_{co}$</td>
<td>Unconfined concrete strength which is equal to 0.85 $f'_c$</td>
</tr>
<tr>
<td>$f_f$</td>
<td>Tensile strength in FRP tube</td>
</tr>
</tbody>
</table>
\( f_{fu} = \) Ultimate tensile strength in FRP tube

\( f_l = \) Lateral confining pressure in FRP tube

\( f_{l,a} = \) Actual confining pressure in FRP tube

\( f_{ls} = \) Confining pressure from transverse steel helix reinforcement

\( f_{yh} = \) Yield tensile strength of transverse steel helix reinforcement

\( f_{yt} = \) Yield tensile strength of the internal tube (PVC or steel)

\( f_{ys} = \) Yield tensile strength of steel bar reinforcement

\( K_N = \) Normalized confinement stiffness

\( L = \) Clear span length of the specimens under four-point bending

\( M (L) = \) The total moment response of the layers

\( M_1 = \) Primary bending moment

\( M_2 = \) Secondary bending moment

\( M_R = \) Total response of the bending moment

\( n_{layers} = \) Number of layers in layer-by-layer numerical integration approach

\( n_s = \) Number of longitudinal steel bars reinforcement

\( P = \) Applied axial load

\( P_R = \) Total response of the axial load

\( r_s = \) Radius from centre of the cross-section to centre of steel bar

\( t_f = \) Thickness of FRP tube
\( t_{f,h} \) = Thickness of FRP tube in the hoop direction

\( t_{f,l} \) = Thickness of FRP tube in the longitudinal direction

\( t_t \) = Thickness of the internal tube (PVC or steel)

\( \beta \) = Coefficient to account for the different confinement mechanism in hollow columns

\( \Delta h \) = Thickness of single layer in layer-by-layer numerical integration approach

\( \varnothing \) = Void ratio \((D_l / D_o)\)

\( \delta \) = Lateral deformation corresponding to the maximum axial load

\( \varepsilon_{cc} \) = Confined concrete compressive strain corresponding to \( f'_{cc} \)

\( \varepsilon_{co} \) = Unconfined concrete compressive strain corresponding to \( f'_{co} \)

\( \varepsilon_{cu} \) = Unconfined concrete compressive strain at ultimate state

\( \varepsilon_{h,rup} \) = Value of ultimate strain at the rupture of FRP tube

\( \varepsilon_f \) = Tensile strain in FRP tube

\( \varepsilon_{fu} \) = Ultimate tensile strain of FRP tube

\( \varepsilon_s \) = Yield tensile strain of the longitudinal steel bars reinforcement

\( \varepsilon_t \) = Yield strain of the internal tube (PVC or steel)
1 INTRODUCTION

1.1 Preamble

Concrete has been widely used as a construction material over the past century. During that time, concrete industry witnessed a considerable progress to improve properties of concrete in both fresh and hardened states. In general, strength, durability and economy of concrete production have been targeted to improve the properties of concrete in different structural applications. In terms of minimizing the construction cost, different techniques are available for structural designers to use. For example, using of hollow concrete sections can provide a reasonable reduction in cost and self-weight of the structure. The main advantage of using hollow reinforced vertical members is to enhance the structural performance of the strength/mass and stiffness/mass ratios. For hollow core concrete columns, a Carbon-Fibre-Reinforced polymer (CFRP) tube can be used to compensate the reduction in the ultimate axial load, which is caused by the existence of an inner hole within the columns’ cross section. In order to obtain further understanding of the behaviour of FRP confined hollow core concrete columns, a part of this study experimentally examines the behaviour of hollow core circular concrete specimens with and without PVC tube.

Also, by using high strength concrete, the cross-sectional area of the concrete members can be reduced considerably. This type of concrete has been widely used in high-rise structures, particularly in columns. However, by increasing the concrete strength it becomes a more brittle material that shows sudden failure mode which is not a preferable structural behaviour. Subsequently, a new type of concrete had been created to achieve concrete that shows ultra-high strength and enhanced ductility compared to normal concrete. This type of concrete is called Reactive Powder Concrete (RPC).
Therefore, this study investigates the experimental and analytical behaviour of sixteen hollow core RPC specimens confined with CFRP tubes. To improve the behaviour of the CFRP-confined hollow core RPC specimens, internal tubes of PVC and steel were used. In this study, specimens were tested under concentric, eccentric and bending loading conditions.

1.2 Overview

Reactive Powder Concrete (RPC) is a new type of composite material which presents very high strength with superior mechanical properties in comparison with normal concrete. The typical ingredients of RPC are general purpose cement, silica fume, fine sand (less than 600 μm), superplasticizer, water and steel fibre. Because of using high quantity of fine sand as a replacement of coarse aggregate, the RPC is produced with a high quantity of cement (900-1000 kg/m³) in order to be sufficient to cover the whole surface area of the fine sand. This kind of concrete mixture was developed by Richard and Cheyrezy (1994).

The RPC is not only characterised by its ultra-high strength but also it has enhanced mechanical and physical properties such as high flexural strength due to using of steel fibre, low permeability, high resistance to corrosion and limited volume changes (Richard and Cheyrezy, 1994). All these desired features in this kind of ultra-high performance concrete will encourage specialists in the civil engineering field to use RPC in construction projects. In addition, RPC can be used where thin concrete sections are required which also presents an advantage of self-weight reduction and minimizes the applied loads from the structure to the foundation.
Hollow concrete columns have been widely used when low weight and economical design is required as a result of reducing the amount of concrete in the structural members. Unfortunately, the current international design codes do not address any particular issue regarding hollow concrete sections. In the construction field, hollow sections have been used in columns and piers of concrete bridges. The reason behind using this kind of sections is to increase the structural performance, strength to mass ratio and stiffness to mass ratio of columns.

In the last few decades, using Fibre Reinforced Polymer (FRP) materials in concrete retrofitting is preferred to steel because of its lower self-weight, higher resistance to aggressive environment and ease of installation (Priestly et al. 1996). The FRP material is commercially available in different types such as Carbon Fibre Reinforced Polymer (CFRP), Glass Fibre Reinforced Polymer (GFRP) and Aramid Fibre Reinforced Polymer (AFRP). These FRP materials have been commonly used in strengthening and repair of reinforced concrete, steel, timber and masonry structures. Flexural and shear strengthening of beams and slabs, increasing load carrying capacity and ductility enhancement of columns are the most popular reasons behind using FRP materials.

Accordingly, a significant self-weight reduction of concrete columns can be achieved by using different techniques such as very high strength concrete (RPC), hollow core section and FRP confinement.

1.3 Scope

In the last three decades, many studies have been conducted to examine the behaviour of concrete columns that are externally confined with FRP material. The majority of these studies were performed on concrete columns with solid sections, while hollow
concrete sections had a limited research focus. In hollow circular columns, the annular concrete section is subjected to non-uniform confining pressure over its radius, and its behaviour can be different from FRP-confined concrete in a solid circular column where the confinement is generally uniform over the cross-section. To minimize the detrimental effects of the inner void, existing studies have explored the use of an additional inner steel tube, leading to the so-called double-skin tubular columns (DSTCs) which was a new type of hollow core concrete section (e.g. Fam and Rizkalla 2001; Teng et al. 2007; Yu et al. 2010b). Also, the high compressive strength characteristic of RPC columns and the effect of steel fibre on the stress-strain behaviour are not fully understood. Thus, this study is dictated to investigate the structural behaviour of hollow core RPC columns.

As a result of the experimental program possibilities that consist of columns’ configuration, concrete type and loading conditions, this research study is limited to:

- The hollow concrete columns are reinforced with only one steel layer in both the longitudinal and transversal directions due to the small cross-sectional dimensions of the hollow column.
- The RPC with steel fibre is designed to a compressive strength of not more than 120 MPa at 28 days due to limitations of testing machine loading capacity.
- All columns are tested under monotonic load only; dynamic load and concrete volume changes such as creep and shrinkage are not included in the experimental program of this study.
- Confinement methods are limited to only two types; CFRP external tube, steel helix and steel and PVC tubes within the concrete section.
- Only hollow concrete columns with a circular section are used to provide a uniform allocation of CFRP confinement.
1.4 Objectives

The main objectives of this study are as follows:

- Investigate the behaviour of circular hollow columns made of RPC and subjected to various modes of monotonic load (concentric, eccentric and flexural).
- Provide a comparison between the behaviour of RPC circular hollow concrete columns with and without steel fibre.
- Demonstrate the efficiency of replacing ordinary steel reinforcement with CFRP tube in terms of strength and ductility of RPC hollow columns.
- Investigate the effect of various static loading mode (concentric, eccentric and flexural) on the behaviour of RPC hollow column confined with CFRP tube.

1.5 Thesis outline

Eight chapters are included in this study, an overview about RPC, hollow columns and FRP confinement in structural members is presented above (Chapter 1). In addition, this chapter shows the significance and objectives of the study.

Chapter 2 presents a review of literature about the principles of RPC, materials involved, mechanical properties of RPC columns and the applications of RPC. In addition, Chapter 2 presents the confined stress-strain behaviour of deferent types of concrete columns such as; confined solid concrete column, confined hollow concrete columns and double skin tubular columns. This chapter also presents a number of the most cited confinement models of concrete columns. At the end of Chapter 2, a summary that addresses the research gap of this study is presented.
In Chapter 3, the test results of an experimental study that has been done to investigate the mechanical properties of the RPC that are discussed in Chapter 2. The main focus of Chapter 3 is on the tensile strength of the RPC.

Chapter 4 shows an experimental preliminary study that has been done to investigate the behaviour of CFRP confined circular hollow concrete specimens with inner PVC tube.

In Chapter 5, an experimental program was conducted based on the results of the preliminary study of Chapter 4. Chapter 6 also includes materials’ properties, columns’ fabrication, testing procedures and test results of hollow core Reactive Powder Concrete (RPC) circular specimens confined with a CFRP tube.

In Chapter 6, the analytical axial load-bending moment diagrams of RPC hollow column are presented. Discussions and Comparisons between the analytical results of and the experimental results of Chapter 5 are also presented in Chapter 6.

Chapter 7 presents a summary and concluding remarks based on the outcomes of each chapter in this study. In addition, recommendations for further research studies are presented in Chapter 7.
2 REVIEW OF LITERATURE

2.1 Introduction

This chapter consists of two parts of review of literature, the first part presents a brief description of the principles that Reactive Powder Concrete (RPC) is based on. Also, details are given in this chapter about materials’ properties (cement, silica fume, fine sand, superplasticizer and steel fibre) that are used in the production of RPC and their effects on the mechanical properties of RPC. The structural behaviour of the RPC, especially in columns, is also discussed and explained based on the existing literature.

In the second part of this chapter, the behaviour of FRP confined concrete columns with different section configurations is presented. These section configurations are solid concrete columns, hollow core concrete columns and double skin (hybrid FRP-concrete-steel) concrete columns. Also, a number of FRP confinement models from the literature are reviewed. At the end of this chapter, general remarks and summary of this literature review are provided.

2.2 Principles of the RPC

The RPC is a relatively new type of ultra-high performance concrete characterized by its ultra-high strength, low permeability and high ductility. The behaviour of the RPC highly depends on the materials type selection, the mix proportion and the quality control of the production. For example, the properties of RPC such as strength, durability and bond between concrete and steel reinforcement are significantly improved by using a high content of binder (cement and silica fume). Because these materials are the source of calcium silicate hydrates which is primarily responsible for the concrete strength (Philippot et al. 1996). Whereas the high ductility and the energy
The absorption of the RPC are due to the presence of a significant amount of high tensile strength steel fibre (about 2000 MPa) within the composition of the RPC (Richard and Cheyrezy, 1995). Figure 2.1 shows typical mix proportions of the materials that are used in the production of the RPC according to Gowripalan et al. (2003).

Compared to conventional concrete, the RPC presents more homogeneity between the components due to the elimination of the course aggregate and this minimizes the differential tensile strain and maximizes the load carrying capacity of the RPC structural members (Richard and Cheyrezy 1995).

The RPC can be produced by using very fine sand with a maximum size of 600 μm, ordinary cement with particles size ranging from 10 to 100 μm and silica fume which has the smallest particle size of 0.1 μm. Thus, the RPC consists of fine particles of an almost similar size which improves the homogeneity of the composite material and minimizes the volume of voids within its structure. For this reason, both the durability and strength of the RPC are increased (Philippot et al. 1996).
The water to binder (cement and silica fume) ratio is one of the key factors that is used to produce the RPC. A full hydration of the Portland cement can be accomplished by water to binder ratio of 0.23. Thus, water to binder ratio is kept low (0.15 to 0.25) in order to assure that there is no excess water within the concrete mix that may cause a reduction of the compressive strength (Richard and Cheyrezy 1995). On the other hand, concrete with very low water to binder ratio is more likely to get loss of workability during fresh state which causes a significant drop in concrete strength. Thus, water reducing admixtures must be used such as superplasticizers in order to keep the desired workability of the RPC with very low water to binder ratio.

2.2.1 Materials properties

2.2.1.1 Cement

Due to the very high cement factor, the choice of cement type and its properties can be an important factor in the performance of the RPC. As a result of the high water demand of the RPC due to the high fine materials content, some types of cement are not recommended to be used in the mixes of RPC. The controlling factors in the selection of the cement type are requirements for strength and durability of the RPC. The strength development of any type of cement highly depends on its constituents. The calcium silicates (C₃S and C₂S) have the main effect on the strength development of the hydrated cement. However, the calcium aluminate (C₃A) participates in the strength development at an early age (the first three days) of cement hydration. The presence of the C₃A is not preferable because it can affect the long term durability of the hydrated cement paste (Neville and Brooks 1997). For this reason, some types of cement, such as high early strength cement are not recommended in the production of the RPC mixes (Gowripalan et al. 1999). On the other hand, Sulfate Resistance Portland Cement is
recommended to be used in the mix design of the RPC to obtain the required strength and durability which consists of high content of C3S and C2S and low content of C3A (Richard & Cheyrezy 1994). However, general purpose cement can also be used to reduce the production cost of the RPC.

2.2.1.2 Silica fume

One of the famed industrial by-products is silica fume (SiO$_4$), which has been commonly used in the mix design of high strength concrete due to its developed pozzolanic properties (Neville and Brooks 1997). Silica fume consists of very fine particles that are able to fill the voids between the cement particles leading to interrupted voids within the concrete matrix (Bonneau et al. 2000). Silica fume also improves the hydration process of the cement by raising the quantity of the calcium silicate hydrate leading to more reduction in the size of the voids within the concrete matrix. A silica fume proportion between 20 to 30 percent of the cement was reported as optimum to be used in the mix design of the RPC (Chan and Chu 2004).

2.2.1.3 Fine sand

In the mix design of the RPC, the coarse aggregate is replaced by fine sand of particle size less than 600 μm. For the production of the RPC, it is preferable to use either silica sand or quartz sand as fine sand. Both strength and durability of the RPC can significantly improve by eliminating pores and increasing particle packing within the matrix of the RPC. The particle size distribution should fall in the range of 150-600 μm. The upper limit is determined by the homogeneity requirements and the lower limit is set to avoid interference with the largest particles of cement. As a result of this
homogeneity of materials, the density is increased thus reducing the permeability of the
RPC leading to enhanced durability (Richard & Cheyrezy 1995).

2.2.1.4 Water content and superplasticizer

To optimize the performance of RPC, water to binder ratio is recommended to be at
minimum levels. According to Neville and Brooks (2010), a water to cement ratio of
0.23 is sufficient to achieve full hydration of cement within the concrete mix. The
negative effect of excess water on the concrete strength is well known. Chemically
uncombined water in the cement hydration process weakens concrete in terms of
strength and durability. For these reasons, RPC is produced with low water to binder
ratio of approximately 0.15 to 0.25 to avoid any excess water (Coppola et al. 1997).
The very low water to binder ratio used in RPC is only possible because of the
fluidizing power of high-quality third generation superplasticizer (Coppola et al. 1997).
Thus, the production of RPC would not be possible without the use of superplasticizer.
Collepardi et al. (2003) investigated the influence of three types of superplasticizer on
the performance of RPC in terms of water to binder ratio and compressive strength.
They concluded that the Acrylic Polymer based superplasticizer performed better than
the Sulfonated Melamine-Formaldehyde (SMF) and Sulfonated Naphthalene-
Formaldehyde (SNF) based superplasticizers in terms of obtaining low water to binder
ratio, regardless the cement and silica fume type used in the production of the RPC
mixes. Collepardi et al. (2003) also concluded that at the RPC mixes with Acrylic
Polymer superplasticizer showed higher compressive strength than the RPC mixes with
SMF or SNF superplasticizers at later ages, regardless of the type of cement and silica
fume.
2.2.1.5 Steel fibre

Due to the mixture of very fine materials, the RPC tends to show a very brittle failure behaviour which is undesirable in structural applications. For this reason, steel fibres are used to increase the ductility and to improve the fracture toughness of the RPC. The main properties of steel fibre that affect the behaviour of RPC are tensile strength, toughness and its bond characteristics with the surrounding concrete. The aspect ratio (length/diameter) of steel fibre is the main factor that influences the bond strength between steel fibres and concrete.

Typically, macro steel fibre are used in the mixture of the RPC having the dimensions of 0.18-0.2 mm in diameter, 12-13 mm in length. The ductility of RPC is significantly affected by the amount and type of steel fibre that is used within the RPC mix. Dugat et al. (1996) reported that the optimum percentage of steel fibres for RPC is between 2 to 3 percent by the total volume of concrete. The influence of the various shapes and sizes of steel fibres was studied by Collepardi et al. (2003). In terms of steel fibre shape: waved, hooked ends and deformed surface fibres are preferred more than straight smooth surface fibres having the same fibre length due to the improvement of bond strength and pullout resistance.

2.2.2 Mixing procedure of RPC

The standard mixing procedure for normal concrete that is described in AS1012.2 (1994) may not be suitable for RPC due to the very fine materials and the very low water to binder ratio used in this type of concrete. The mixing procedure of RPC mainly depends on the type of the mixer. The higher energy of the mixer, the better distribution of fine ingredients in a mixture of RPC. The RPC was produced with different types of mixers including; central mixer, truck mixer and laboratory mixer. A mixing procedure
of RPC for laboratory mixers was suggested by Bonneau et al. (2000), as shown in Figure 2.2.

Figure 2.2 Mixing procedure of RPC (Bonneau et al. 1997)

### 2.2.3 Mechanical properties of RPC

The RPC exhibits very high strength and durability properties compared with normal concrete. Steel fibres added to the mix of RPC in order to increase concrete ductility and flexural strength (Cheyrezy et al. 1994). The behaviour of RPC is highly affected by its ingredients’ type and content. According to previous studies (Richard & Cheyrezy 1994; Dugat et al. 1996; Bonneau et al. 2000; Voo et al. 2001; Gowripalan et al. 2003; Graybeal 2006), the compressive strength varied from 160 to 197 MPa, the flexural
strength varied from 25 to 50 MPa, the indirect tensile strength (splitting strength test) varied from 12 to 21 MPa and the elastic modulus varied from 44 to 62 GPa.

The mechanical properties of RPC are significantly improved by obtaining a maximum packing of its materials particles. In addition, the chemical reaction between the hydrated Portland cement compounds and the silica fume produces a very dense microstructure and thus improves the bond between the aggregate and the surrounding cement paste (Shihada & Arafa 2010).

2.2.4 Structural behaviour of RPC

Several studies have been conducted on the production of RPC which also investigated the mechanical properties and durability of this type of concrete. However, a limited number of studies were found in the literature which examined the structural behaviour of RPC members. In this section, attention is given to the studies conducted on RPC columns.

Malik and Foster (2010) conducted an experimental study on RPC columns. An axial stress-strain curve was obtained of axially loaded RPC specimens reinforced with 2% of steel fibre. The experimental program consisted of two phases. In the first phase, a group of six columns were made of steel fibre reinforced RPC with a compressive strength of 150 MPa. The RPC columns were reinforced with longitudinal steel reinforcement, however, no lateral reinforcement was used in the middle third of the columns’ height. It was reported that the addition of steel fibres to RPC significantly increases the compressive strength and slightly increases the modulus of elasticity. Also, based on the experimental results of the study, Malik and Foster (2010) concluded that the number of ties can be considerably reduced in RPC columns when sufficient amount of steel fibres is used.
In the second phase of the study, Malik and Foster (2010) investigated the behaviour of sixteen CFRP confined columns made of RPC with a compressive strength of 160 MPa. No steel reinforcement was used; however, half of the columns were reinforced with 2% of steel fibres and the rest were without steel fibres. The columns were loaded concentrically and eccentrically up to failure. Based on the test results of concentrically loaded columns, the CFRP confined RPC columns exhibited an increment of 19% in the load carrying capacity compared with the unconfined RPC columns. For eccentrically loaded columns, it was found that the CFRP confinement can improve the ductility of RPC columns by exhibiting a considerable straining after the maximum stress point.

Zhao and Hao (2010) investigated the seismic behaviour of two rectangular hollow columns. These columns were made of RPC with a compressive strength of 140 MPa and tested under cyclic horizontal load. Both columns had the same material properties and geometry; however, one of the columns was reinforced with double the amount of transverse steel reinforcement. Based on the experimental results, both columns showed an accepted seismic behaviour. Zhao and Hao (2010) also reported that the increase of the amount of transverse reinforcement can enhance the ductility response of RPC hollow columns.

Zheng et al. (2012) experimentally examined the compressive stress-strain behaviour of RPC with three different percentages of steel fibre (1%, 2% and 3%) after being subjected to elevated temperature (20°C to 900 °C). Zheng et al. (2012) concluded that both strength and ductility can be enhanced by increasing steel fibres’ content. They also stated that the 2% of steel fibres content is the optimum volumetric content because no significant improvement was achieved for the properties of RPC post the 2% of steel fibres in an economical point of view.
Zohrevand and Mirmiran (2012) examined the lateral cyclic behaviour of GFRP confined RPC columns. The experimental program consisted of four half scale columns. The first two columns were reinforced with conventional steel bars, one of them was made of normal concrete of 50 MPa and the other made of RPC of 150 MPa. The second two columns were concrete filled GFRP tube columns and also two types of concrete were used to fill the GFRP tubes; normal concrete (50 MPa) and RPC (150 MPa). All columns were subjected to a monotonic axial load and increasing cyclic lateral load. Based on the test results, there were no differences in the carrying load capacity and axial deformation between conventional steel reinforcement and GFRP columns. It was found that the RPC filled GFRP tubes showed reasonable energy absorption and ductility with no conventional steel reinforcement being used.

2.2.5 Applications of RPC

In the last two decades, structural applications of RPC gained progress in the construction field as a number of projects were completed using this type of concrete. The first structural project of RPC in the world was Sherbrooke foot-bridge in Canada, that was constructed in 1997 (Adeline et al. 1998). In this project, the RPC was designed to have a compressive strength of 200 MPa and filled in steel tubes to form composite segments in a 60 meter post-tensioned bridge. After that, several structural projects were constructed worldwide using RPC such as Majata footbridge in Japan which was a 51 meter prestressed span, made of pre-cast RPC elements. Another project was Peace footbridge in South Korea with a 120 meter span made of post tensioned segments. In 2003, Australia had the first RPC bridge in the world which was opened for normal highway traffic (Rebentrost 2006). Shepherds Creek Bridge was constructed with a span of 15 meter and a width of 21 meter. Later in 2006, Papatoetoe footbridge in Auckland, New Zealand was opened to the public. This bridge consisted of 10 spans
that are simply supported at the ends of each span, the total length of the footbridge is 175 metre, (Rebentrost 2006).

2.3 External confinement of concrete columns

The structural performance of the concrete columns can be improved significantly with confinement techniques. Both the load carrying capacity and the ductility of concrete columns are enhanced due to the effect of confinement. Using Fibre Reinforced Polymer (FRP) materials in concrete confinement is preferred to steel because of its lower self-weight and higher resistance to aggressive environment, such as corrosion. In following sections, the behaviour of FRP confined concrete columns with different section configurations is presented. These section configurations are solid concrete columns, hollow core concrete columns and double skin (hybrid FRP-concrete-steel) concrete columns. Also, a number of FRP confinement models from the literature are reviewed. Finally, general remarks and summary of this literature review are provided.

2.3.1 Behaviour of FRP confined concrete columns

Lateral confinement of reinforced concrete columns is used in order to inhibit the lateral expansion due to Poisson’s effect. This confinement can improve the axial load behaviour and ductility of concrete columns. For this purpose, steel ties or steel helixes can be used in the transverse direction along the length of the column. Both the strength and ductility of the columns are enhanced with this type of lateral confinement.

In the last few decades, many studies were conducted on using FRP material as a lateral confinement of reinforced concrete columns. External confinement with FRP materials is preferred for its low weight, high corrosion resistance and reliable tensile strength. The FRP material that is used in confinement is available as a sheet and tube. Both of
these types have effective confinement performance of reinforced concrete columns. Figure 2.3 shows different types of FRP materials used in columns confinement.

![Figure 2.3 FRP in different types; (a) CFRP and GFRP sheet (b) CFRP and GFRP tube.](image)

It is agreed that holding the lateral strain of the concrete can significantly increase both the ultimate strength and ductility of concrete as a result of confinement. One of these confinement methods, as mentioned above, is wrapping a concrete column with FRP sheets. For this purpose, FRP sheets should be bonded to the exterior surface of the concrete column and oriented in the hoop direction to provide maximum confinement for the concrete. Several experimental studies (e.g., Nanni and Bradford 1995; Fam and Rizkalla 2001; Lam and Teng, 2003; Hadi 2006; Hadi 2009) have confirmed the bilinear stress-strain behaviour of FRP-confined concrete.

Because of the linear stress-strain behaviour of FRP material, the load carrying capacity of the confined concrete continues to increase up to the failure of FRP material. In general, FRP tube may fail in two modes: (1) by FRP tube rupture in the hoop direction as it reaches the ultimate tensile strength; or (2) by FRP tube de-bonding at the overlapping area. Nanni and Bradford (1995) and Samaan et al. (1998) stated that the
FRP starts its effect on the stress-strain behaviour once the concrete reaches its ultimate strength and with an axial strain of approximately 0.003. It is important to mention that confinement effectiveness highly relies on the FRP stiffness and concrete dimensions.

For FRP-confined circular concrete columns under axial compression, a lateral expansion is produced that results in tension stresses through the FRP in the direction of the circumference, as shown in Figure 2.4. The tension stress creates a uniform lateral pressure ($f_l$) that continues to increase as the applied axial load increases until the failure of the concrete and the FRP is reached.

![Figure 2.4 Confining action of FRP on concrete core: (a) FRP; (b) concrete core (Ozbakkaloglu and Lim 2013)](image)

According to the compatibility of the deformation between the concrete surface and the FRP, the lateral pressure ($f_l$) can be calculated by using Equation 2.1. Also, based on previous studies (Pessiki et al. 2001; Lam and Teng 2003; Lorenzis and Tepfers 2003; Ozbakkaloglu and Oehlers 2008), the value of ultimate strain at the rupture of FRP sheet ($\varepsilon_{h,rup}$) is lower than the value of ultimate tensile strain ($\varepsilon_{f_u}$) of FRP material. Thus, Pessiki et al. (2001) suggest a reduction factor ($k_\varepsilon$) to be used in the calculation of the lateral pressure ($f_l$) as shown in Equation 2.2. For the same reason, Lam and Teng
(2003) introduced the actual confining pressure \( f_{l,a} \) in Equation 2.3 to account for the premature failure of the FRP confining system.

\[
f_l = \frac{2E_f t_f \varepsilon_{fu}}{D_o} \tag{2.1}
\]

\[
\varepsilon_{h,rup} = k_e \varepsilon_{fu} \tag{2.2}
\]

\[
f_{l,a} = \frac{2E_f t_f \varepsilon_{h,rup}}{D_o} \tag{2.3}
\]

where, \( f_l \) is the lateral confining pressure of the FRP tube in MPa; \( E_f \) is the modulus of elasticity of FRP material in MPa; \( t_f \) is the thickness of FRP tube in mm; \( D_o \) is the outer diameter of concrete section in mm; \( \varepsilon_{h,rup} \) is the value of ultimate strain at the rupture of FRP tube; \( k_e \) is a reduction factor; \( \varepsilon_{fu} \) is the ultimate tensile strain of FRP material; and \( f_{l,a} \) is the actual confining pressure of the FRP tube in MPa.

### 2.3.2 Behaviour of hollow core concrete columns

For the design of solid concrete columns, there is no ideal behaviour of uniaxial loaded columns. However, understanding this behaviour provides reliable basic information of these theories which are used in the design of concrete columns. Thus, a great number of studies have been conducted in this area of study (e.g., Richart et al. 1928, Mander et al. 1988, Miyauchi et al. 1997, Toutanji 1999, Lam and Teng 2003, Wu et al. 2006). As a result, design codes adopted some of these studies to calculate the longitudinal and the transverse reinforcement which are required to carry the design loads and to provide reliable ductility of concrete columns. However, there is no classification for issues assigned to the design of hollow core concrete columns in the current design codes. Also, only a few studies have been performed in terms of understanding the structural behaviour of hollow concrete columns.
For some structural applications, hollow concrete column as shown in Figure 2.5 is used where the existence of a void within the column’s cross section may cause reduction in the ultimate axial load that the concrete column can carry depending on some factors such as: column’s void ratio (diameter of inner hole / diameter of column), column size and concrete strength. However, compensation of this reduction in the load capacity can be achieved by FRP confinement.

Zahn et al. (1990) investigated the behaviour of full scale concrete columns with hollow circular cross-section with a diameter of 0.4 m and a height of 3.4 m. Six circular hollow columns have been cast with three different void ratios (diameter of inner hole / diameter of column) and subjected to axial load and cyclic flexural load. These columns were reinforced with one layer of longitudinal steel reinforcement near the outer face of the column and laterally confined with helical steel reinforcement. Based on the experimental results, it had been concluded that the hollow columns with low load carrying capacity, moderate steel ratio and low void ratio tend to behave in a ductile manner under the flexural test. On the other hand, hollow columns with high load carrying capacity, high steel ratio and low void ratio tend to behave in a brittle manner under the flexural test. According to the authors, the brittle behaviour of columns came
as a result of the concrete crushing near the unconfined inner face of the section. Regarding the analytical modelling part of the study, the following stress-strain relation was suggested:

\[
\frac{f'_{cc}}{f'_{co}} = \frac{xr}{r-1+x^r}
\]  
(2.4)

\[
x = \frac{\varepsilon_{co}}{\varepsilon_{cc}}
\]  
(2.5)

\[
r = \frac{E_c}{E_c - E_{sec}}
\]  
(2.6)

\[
E_{sec} = \frac{f'_{cc}}{\varepsilon_{cc}}
\]  
(2.7)

\[
\frac{\varepsilon_{cc}}{\varepsilon_{co}} = 1 + R \left( \frac{f'_{cc}}{f'_{co}} - 1 \right)
\]  
(2.8)

where, \( f'_{cc} \) is the confined concrete compressive strength (MPa) and \( f'_{co} \) is the unconfined concrete strength which is equal to 0.85 of the cylinder concrete compressive strength at 28 days (MPa); \( \varepsilon_{cc}, \varepsilon_{co} \) are the compressive strain of concrete corresponding to \( f'_{cc}, f'_{co} \), respectively; \( R \) is an empirical constant.

For hollow core concrete columns, FRP tubes can be used to compensate the reduction in the ultimate axial load, which is caused by the existence of an inner hole within the columns’ cross section. In order to obtain further understanding of the behaviour of FRP confined hollow core concrete columns, a number of studies have been performed. These include, Modarelli et al. (2005); Lignola et al. (2007); Yazici and Hadi (2009); Kusumawardaningsih and Hadi (2010); Yazici and Hadi (2012); and Hadi and Le (2014).

Fam and Rizkalla (2001a, b) conducted experimental and analytical studies on twelve GFRP confined concrete columns, half of them were hollow columns. For the
experimental study, Fam and Rizkalla (2001a) concluded that the FRP confinement effectiveness is influenced by the presence of the inner hole within the section of the concrete column. However, no significant change in column ductility has been noticed according to their results. Fam and Rizkalla (2001a) also concluded that providing the FRP tube inside the hole can enhance the confinement effectiveness to match that of solid confined concrete columns. For the analytical study, Fam and Rizkalla (2001b) suggested an incremental passive confinement model. This model can be used to estimate the axial stress-strain response of both solid and hollow columns based on a model proposed by Mander et al. (1988).

Lingola et al. (2007) conducted an experimental and analytical study on CFRP confined hollow square cross section concrete columns. The strength and ductility behaviour of these columns were investigated under concentric and eccentric loading conditions. It was shown that the strength of hollow columns is increased and the ductility is significantly enhanced.

There are a number of different factors which can affect the behaviour of FRP confined hollow reinforced concrete columns, such as: hollow core size and shape; FRP type and thickness; and concrete type and strength. Kusumawardaningsih and Hadi (2010) studied the shape influence of inner hole on the effectiveness of FRP confinement. It was found that using a circular hollow core had better performance than using a square hollow core in terms of improving the strength and ductility of FRP confined hollow columns.

Yazici and Hadi (2012) suggested a normalized confinement stiffness approach to predict the strength and strain of FRP-confined solid concrete. This simple model was developed based on some modifications to a model proposed by Richart et al. (1928).
Yazici and Hadi (2012) examined and compared the accuracy of their model to the guidelines of the American Concrete Institute using an experimental database available in the literature and the suggested approach showed a reliable prediction for both strength and strain of FRP-confined solid concrete. Also, they extended this approach to be applicable for FRP-confined hollow core concrete. Equations 2.9 and 2.10 were proposed by Yazici and Hadi to predict the stress-strain behaviour of FRP-confined hollow core concrete:

\[
\frac{f'_{cc}}{f'_{co}} = (1 + 0.033 K_N) \beta \tag{2.9}
\]

\[
\frac{\varepsilon_{cc}}{\varepsilon_{co}} = (1 + 0.16 K_N) \beta \tag{2.10}
\]

\[
K_N = \frac{2 E_f t_f}{D f'_{co}} \quad 10 \leq K_N \leq 20 \tag{2.11}
\]

\[
\beta = \left(1 - \frac{D_i^2}{D_o^2}\right) \tag{2.12}
\]

where, \( K_N \) is a normalized confinement stiffness; \( \beta \) is a coefficient to account for the different confinement mechanism in hollow columns; \( D_i \) is the hollow core diameter of the concrete cylinder; \( D_o \) is the diameter of the concrete cylinder.

Hadi and Le (2014) conducted an experimental study to investigate the behaviour of hollow core concrete columns wrapped with CFRP sheets. These columns were wrapped with CFRP sheets in three different combinations of wrapping orientations (0°, 45° and 90° with respect to the circumferential direction). It was found that the strength and ductility of hollow core concrete columns were increased for all wrapping configurations but the increase in the strength was minor. The highest results were
obtained with columns that were exclusively wrapped with CFRP sheets in the hoop direction.

Since there are few studies in the literature dealing with the confinement response of the FRP hollow concrete columns and the design codes of the structural design left this type of columns out of discussion, further research is needed in this field to obtain better understanding to the structural response of hollow concrete columns.

2.3.3 Behaviour of double skin concrete columns

The Double Skin Tubular Column (DSTC) is a new type of hybrid hollow concrete columns. In most cases, an external FRP tube is used while the internal tube is made of steel, as shown in Figure 2.6. The major purpose of using the exterior tube is to increase the axial strength capacity by confining concrete in the hoop direction. While the inner tube takes the role of the steel in the longitudinal direction and also it internally controls the spalling of concrete. Using these three materials (Concrete, steel and FRP) provides structural columns with excellent properties, such as high load carrying capacity, high ductility and corrosion resistance (Teng et al. 2004 and Yu et al. 2010). Recently, several studies have been performed to explain the response of DSTC, such as: Fam and Rizkalla (2001); Teng et al. (2007); Wong et al. (2008); Yu et al. (2010); Ozbakkaloglu and Fanggi (2013).

Among these studies, Fam and Rizkalla (2001) reported tests on FRP-concrete DSTCs with an inner tube made of FRP; Teng et al. (2007) and Yu et al. (2010b), among others, reported tests on FRP-concrete-steel DSTCs with an inner tube made of steel. These studies generally demonstrated that with the additional inner tube, both the performance of the column and the effectiveness of confinement can be significantly improved.
In the existing studies on FRP-confined DSTCs, the inner tubes used were typically stiff and also served as longitudinal reinforcement. However, for hollow columns with a small- or medium-size void, the use of a stiff inner tube is inefficient in resisting bending. In such cases, the main longitudinal reinforcement should be placed away from the inner edge of the concrete section, while the function of the inner tube should be mainly to restrain the inner surface of concrete for effective confinement. As a result, the inner tube could be made of a less stiff material (e.g. PVC) and be more cost-effective than existing solutions. The permanent inner PVC tube has also many other advantages in construction industry besides the low cost such as; excellent durability, ease of fabrication and handling.

![Diagram of double skin tubular column](image)

Figure 2.6 A typical section of double skin tubular column

According to an experimental study conducted by Fam and Rizkalla (2001a) on twelve GFRP confined concrete specimens. It was found that the presence of inner hole reduces the confinement effectiveness while maintaining an acceptable level of ductility within the confined concrete specimen. However, the efficiency of the GFRP confinement can be achieved as the same level as solid concrete specimens by providing inner tube with reliable stiffness. Fam and Rizkalla (2001a) also concluded that the stress-strain
behaviour of the GFRP confined concrete is bilinear and the slope of the line after the transition zone depends on the inner tube stiffness and the size of the inner hole.

Wong et al. (2008) had similar conclusions to those of Fam and Rizkalla (2001a) after testing 43 concrete specimens including 18 specimens of DSTC. These 18 specimens were made of an external FRP tube, an internal steel tube and concrete between them. Wong et al. (2008) concluded that the internal tube has an important role in terms of the inner confinement and improving the stress-strain behaviour of DSTC. In addition, they suggested that a new model is needed to clarify the response of DSTC.

Yu et al. (2010) proposed a simple new model to explain the response of DSTC by employing Finite Element (FE) model and previous experimental results. The FE model has been verified with current test results. Moreover, the effect of a group of parameters on the confinement efficiency was investigated through a parametric study by using the FE model. Yu et al. (2010) recommended using the new model in the design of DSTC as it gives reliable accuracy. Equations 2.13 and 2.14 are proposed by the authors to obtain the confined stress and the ultimate axial strain, respectively.

\[
\frac{f_{ce}}{f_{co}} = \begin{cases} 
1 + 3.5(\rho_K - 0.01)\rho_e & \text{if } \rho_K \geq 0.01 \\
1 & \text{if } \rho_K < 0.01 
\end{cases} \tag{2.13}
\]

\[
\frac{\varepsilon_{ce}}{\varepsilon_{co}} = 1.75 + 6.5 \rho_K^{0.8} \rho_e^{1.45} (1 - \emptyset)^{-0.22} \tag{2.14}
\]

\[
\rho_K = \frac{E_f \cdot t_f}{(E_{sec0} R_o)} \tag{2.15}
\]

\[
\rho_e = \frac{\varepsilon_{h,\text{rup}}}{\varepsilon_{co}} \tag{2.16}
\]

\[
E_{sec0} = \frac{f'_{co}}{(\varepsilon_{co} f_i)} \tag{2.17}
\]
where, $\mathcal{O}$ is the void ratio (in circular columns = diameter of inner hole/diameter of column); $R_o$ is the outer radius of the confined concrete core in mm.

### 2.3.4 Confinement models of concrete columns

The FRP confinement behaviour of the concrete has been studied for more than thirty years. Various models have been suggested by many researchers (e.g. Fardis and Khalili 1981; Mander et al. 1988; Saadatmanesh et al. 1994; Miyauchi et al. 1997; Toutanji 1999; Lam and Teng 2003a; Tamuzs et al. 2006; Wu et al. 2006; Youssef et al. 2007; Xiao et al. 2010; Lim and Ozbakkaloglu 2014). In this section, the research progress regarding the stress-strain behaviour of FRP-confined concrete with circular section is presented. Most of the well-known stress-strain models of FRP-confined concrete are presented, starting with the early trails to create the stress-strain models of FRP-confined concrete and ending with the recent models.

The classification of models that predict the stress-strain response of concrete confined with FRP materials consist of two types of models: the first one is based on steel confined concrete model that was developed by Richart et al. (1928) and Newman and Newman (1969). Studies that adopted this model claimed that this type of model can be modified to suit FRP confined concrete columns; while the second type is based on empirical or analytical study.

Based on an empirical study by Richart et al. (1928) conducted on steel confined concrete specimens with unconfined compressive strength, $f'_c o$, in the range of 20 to 50 MPa, Equations 2.18 and 2.19 have been proposed to predict the confined axial compressive strength, $f'_c c$, and compressive strain, $\varepsilon_{cc}$, respectively.
\[ \frac{f_{cc}}{f_{co}} = 1 + 4.1 \frac{f_l}{f_{co}} \]  
(2.18)

\[ \frac{\varepsilon_{cc}}{\varepsilon_{co}} = 1 + 5 \left( \frac{f_{cc}}{f_{co}} - 1 \right) \]  
(2.19)

An earlier attempt by Fardis and Khalili (1981) employed this kind of model (steel-based model) to clarify the FRP confinement response regarding the maximum strength and strain by involving the lateral pressure \( f_l \), which causes FRP rupture due to the tensile stress in the hoop direction. The maximum lateral pressure can be determined by using Equation 3.16 assuming that the lateral stress is constant.

\[ f_l = \frac{2f_f t_f}{D_o} \]  
(2.20)

In addition, Fardis and Khalili (1981) developed two equations (Equations 2.21 and 2.22) to determine the ultimate confined concrete stress and strain, respectively, as below:

\[ \frac{f_{cc}}{f_{co}} = 1 + 3.7 \left( \frac{f_l}{f_{co}} \right)^{0.86} \]  
(2.21)

\[ \varepsilon_{cc} = \varepsilon_{co} + 0.0005 \left( \frac{E_l}{f_{cc}} \right) \]  
(2.22)

\[ E_l = \frac{2E_f t_f}{D_o} \]  
(2.23)

where, \( E_l \) is the lateral confinement stiffness in MPa.

Fardis and Khalili (1981) used confinement ratios \( (f_l / f_{co}) \) in the range of 0.1 to 0.6 and sensible agreement was detected between the experimental results and the proposed model in terms of the ultimate stress prediction. Fardis and Khalili (1981), however, did not compare the ultimate strain test results with the predicted strain values in their study.
An equation with nonlinear expression between the axial confined strength and the lateral confinement pressure of confined concrete was proposed by Mander et al. (1988). According to Mander et al. (1988), Equation 2.24 was obtained based on tri-axial test results to provide constant pressure for the confined concrete. In comparison with Equation 2.18 that was proposed by Richart et al. (1928), Equation 2.24 has no limitation of concrete grade.

\[
\frac{f'_{cc}}{f'_{co}} = 2.254 \sqrt{1 + 7.94 \frac{f_l}{f'_{co}} - 2 \frac{f_l}{f'_{co}} - 1.254} \quad (2.24)
\]

It was found a reasonable agreement between Equation 2.18 and Equation 2.24 in terms of prediction of the ultimate confined stress, especially with confinement ratio up to 0.7.

Based on the results of an experimental program that consisted of testing concrete cylinders with two different sizes (100 × 200 mm and 150 × 300 mm) wrapped with CFRP sheets, Miyauchi et al. (1997) suggested the equations below to predict the ultimate confined concrete stress and strain:

\[
\frac{f'_{cc}}{f'_{co}} = 1 + 3.485 \left( \frac{f_l}{f'_{co}} \right) \quad (2.25)
\]

\[
\frac{\varepsilon_{cc}}{\varepsilon_{co}} = \begin{cases} 
1 + 10.6 \left( \frac{f_l}{f'_{co}} \right)^{0.373} & (f'_{co} = 30 \text{ MPa}) \\
1 + 10.5 \left( \frac{f_l}{f'_{co}} \right)^{0.525} & (f'_{co} = 50 \text{ MPa})
\end{cases} \quad (2.26)
\]

It was found that Equation 2.25 showed a similar linear pattern of confined strength prediction to Equation 2.18 that obtained by Richart et al. (1928).

Toutanji (1999) suggested a model to estimate the stress-strain behaviour of concrete samples confined with FRP material, depending on experimental and analytical results. Three different types of FRP confinement were used: two layers of CFRP and one layer
of GFRP were used to confine concrete columns with compressive strength of 31 MPa. According to this study, the confinement ratio was in the range of 0.30 to 0.83.

Equations 3.27 and 3.28 represent the prediction models that give estimated values of FRP confined axial strength and ultimate axial strain, respectively. According to Toutanji (1999), these two equations were obtained depending on analysis of test results, assuming that the failure occurs as the hoop strain attains the rupture strain value of FRP confinement, which had been recorded during the coupon test.

\[ \frac{f'_{cc}}{f'_{co}} = 1 + 3.5 \left( \frac{f_t}{f'_{co}} \right)^{0.85} \]  \hspace{1cm} (2.27)

\[ \frac{\varepsilon_{cc}}{\varepsilon_{co}} = 1 + \left( 310.57 \varepsilon_{fu} + 1.90 \right) \left( \frac{f'_{cc}}{f'_{co}} - 1 \right) \]  \hspace{1cm} (2.28)

By using a considerable number of experimental test results (76 FRP-confined concrete specimens), Lam and Teng (2003a) obtained a simple model to predict the FRP confined axial strength for different types of FRP material. This simple design-oriented model explained the bi-linear stress-strain behaviour of FRP-confined concrete starting with parabolic ascending branch followed by linear-elastic ascending branch. They concluded that the ultimate tensile strain of the FRP material should not be used to predict the stress-strain response of the FRP-confined concrete. Alternatively, the actual rupture strain in the hoop direction can be used to show more accurate prediction for the stress-strain behaviour. In addition, Lam and Teng (2003a) demonstrated in this model how the ultimate status of confined columns with FRP can be affected by the jacket strain capacity. Equations 2.29 and 2.30 were suggested by Lam and Teng (2003a) to determine both of the confined concrete compressive strength \( \left( f'_{cc} \right) \) and strain \( \left( \varepsilon_{cc} \right) \), respectively.

This model was adopted by the ACI Committee 440.2R (2008).
\[ f'_{cc} = f'_c \left( 1 + 3.3 \frac{f_{la}}{f'_c} \right) \]  

(2.29)

\[ \varepsilon_{cc} = \varepsilon_{co} \left( 1.75 + 12 \left( \frac{f_{la}}{f'_c} \right) \left( \frac{\varepsilon_{hrp}}{\varepsilon_{co}} \right)^{0.45} \right) \]

(3.30)

where, \( f_{la} \) is the actual confining pressure from FRP confinement.

Wu et al. (2006) suggested a number of confinement models to estimate the axial stress and strain behaviour of FRP confined concrete columns based on an experimental study. The experimental matrix consisted of 300 concrete columns that have unconfined compressive strengths between 23 to 75.4 MPa and confined with different types of FRP materials (GFRP, CFRP and AFRP).

In addition, the confinement ratio varied from 0.047 to 0.28 and depending on the degree of confinement, the axial stress-strain behaviour was divided in this study into two categories: hardening and softening.

According to Wu et al. (2006), this behaviour can be determined based on boundary value (\( \lambda \)), which is equal to 0.13 for FRP sheets with normal modulus of elasticity \( E_f \leq 250 \text{ GPa} \), while it is equal to 0.13\((250/E_f)^{0.5} \) for FRP with high modulus of elasticity. Thus, if the value of confinement ratio \( (f_l/f'_{co}) \) is higher than the boundary value, the stress-strain has a hardening behaviour and if it is not, then the behaviour is considered softening. After this stage, Wu et al. (2006) suggested a number of equations presented below in order to predict axial stress-strain behaviour of FRP confined concrete columns for hardening and softening stress-strain behaviour:

- Hardening behaviour

\[ \frac{f'_{cc}}{f'_{co}} = 2 \left( \frac{f_l}{f'_{co}} \right) + 1 \quad \text{(FRP wrap with normal modulus)} \]

(2.31)
\[
\frac{f'_{cc}}{f'_{co}} = 2.4 \left( \frac{f_{l}}{f'_{co}} \right) + 1 \quad \text{(FRP wrap with high modulus)} \tag{2.32}
\]
\[
\varepsilon_{cc} = 1.785 \varepsilon_{fu} \left( \frac{f_{l}}{f'_{co}} \right)^{1.515} \quad \text{(FRP wrap with normal modulus)} \tag{2.33}
\]
\[
\varepsilon_{cc} = 1.785 \frac{\varepsilon_{fu}}{k_1} \left( \frac{f_{l}}{f'_{co}} \right)^{1.515} \quad \text{(FRP wrap with high modulus)} \tag{2.34}
\]

where, \( k_1 = \sqrt{250/E_f} \) and \( E_f \) in GPa and \( \geq 250 \).

- Softening behaviour

\[
f'_{cc} = f'_{co} \left( 1 + 0.002 \frac{30}{f'_{co}} \frac{\rho_f E_f}{\sqrt{f'_{co}}} \right) \tag{2.35}
\]
\[
\varepsilon_{cc} = \varepsilon_{co} \left( 1 + 0.007 \frac{30}{f'_{co}} \frac{\rho_f E_f}{\sqrt{f'_{co}}} \right) \tag{2.36}
\]
\[
f'_{cc} = f'_{co} \left( 0.75 + 2.5 \frac{f_{l}}{f'_{co}} \right) \tag{2.37}
\]
\[
\varepsilon_{cc} = \varepsilon_{co} \left( 1.3 + 6.3 \frac{f_{l}}{f'_{co}} \right) \tag{2.38}
\]

where, \( \varepsilon_{co} \) is the unconfined compressive strain of concrete which can be taken 0.0038.

Based on an experimental study, Tamužs et al. (2006) proposed a model to predict the ultimate axial strain of CFRP-confined concrete specimens. The nominal compressive strength of the concrete ranged between 20 MPa to 60 MPa. Two different sizes of cylindrical specimens were used in the study, 150 \( \times \) 450 mm and 250 \( \times \) 750 mm (diameter \( \times \) height). The 150 mm diameter specimens were confined with 3, 5 and 7 layers of CFRP confinement and the 250 mm diameter specimens were confined with 5 layers of CFRP confinement. Equation 2.39 below was recommended by Tamužs et al. (2006) to be used for predicting the ultimate axial strain.
Youssef et al. (2007) suggested a model to estimate the behaviour of concrete columns confined with FRP materials. Youssef et al. (2007) tested columns with a large-scale and three different shapes of cross sections; circular, square and rectangular, in addition to concrete specimens 152 × 305 mm. The total number of specimens was 87 concrete columns confined with two kinds of FRP (CFRP and GFRP) and tested under axial compression load. The unconfined concrete compressive strength varied from 27.6 to 34.5 MPa. Youssef et al. (2007) proposed Equations 2.40 and 2.41 to predict the ultimate axial stress and strain of circular concrete columns confined with FRP:

\[
\frac{f_{cc}}{f_{co}} = 1 + 2.25 \left( \frac{f_l}{f_{co}} \right)^{5/4}
\] (2.40)

\[
\varepsilon_{cc} = 0.003368 + 0.259 \left( \frac{f_l}{f_{co}} \right) \left( \frac{f_f}{E_f} \right)^{2/3}
\] (2.41)

Xiao et al. (2010) studied the behaviour and modelling of FRP-confined normal strength and high strength concrete. For high strength concrete (HSC), it was concluded that the model proposed by Jiang and Teng (2007) showed an accurate stress-strain prediction of FRP-confined HSC. Equations 2.42 and 2.43 were suggested to model the stress-strain behaviour of FRP-confined HSC:

\[
\frac{f_{cc}}{f_{co}} = 1 + 3.5 \frac{f_l}{f_{co}}
\] (2.42)

\[
\frac{\varepsilon_{cc}}{\varepsilon_{co}} = 1 + 17.5 \left( \frac{f_l}{f_{co}} \right)^{1.2}
\] (2.43)

2.3.4.1 General remarks on the models of FRP confinement
Ozbakkaloglu et al. (2011) presented a study to review and assess 88 models. The main purpose of the study was to predict the stress-strain response of circular concrete sections confined with FRP material. Ozbakkaloglu et al. (2011) divided the models into two types; Design Oriented Models and Analysis Oriented Models. The majority of the models were created depending on experimental results which were classified as design oriented models. Based on a statistical assessment to investigate the accuracy of prediction of these models, Ozbakkaloglu et al. (2011) concluded the following points:

- More accuracy is expected when using the Design Oriented Models rather than Analysis Oriented Models in terms of prediction the ultimate confined stress and strain, especially with increasing the number of experimental data.
- It has been concluded that the model suggested by Lam and Teng (2003) shows the most accurate prediction of the ultimate confined stress. While the model suggested by Tamuzs et al. (2006) is the most precise model in terms of prediction the ultimate confined strain.
- A higher reliability is expected with models that used hoop strain than the models that used the tensile strain of FRP material directly.
- The prediction’s accuracy of the ultimate stress is considerably higher than the ultimate strain. This is because most of the models are unable to predict the effect of strain sensitivity to the type of FRP material.

2.4 Summary

To summarize this chapter, it was found that there are many research studies carried out on the production of RPC which is an ultra-high performance concrete, but few studies have been published on the structural behaviour of RPC members, especially FRP confined RPC columns. According to the literature, RPC can be prepared and mixed in
laboratory apparatus with acceptable mechanical properties. In terms of structural behaviour, the RPC without fibre is very brittle material and to increase the ductility of the RPC, steel fibres must be added. The ductility of RPC structural members such as columns can be significantly enhanced by using FRP confinement.

In addition, there is a reliable number of FRP confinement models that have been suggested by previous studies to explain the stress-strain response of concrete columns. The models shown in Appendix A are the most cited models in the relevant studies. The Design Oriented Models which were obtained based on experimental investigations were commonly adopted by the researchers. The accuracy of these types of models can be improved by increasing the size of the database of the experimental test results. In addition, using a large size of test samples in order to simulate the test conditions with the real structural conditions of the concrete columns, may lead to better accuracy for these models. It is also important to mention that most of the models illustrated above are proposed for solid confined concrete columns. More research studies are required to investigate the behaviour of FRP confined RPC columns.

The next chapter of this study is dedicated to produce and investigate the mechanical properties of the RPC. The tensile strength of the RPC is
3 EXPERIMENTAL STUDY ON PRODUCTION AND PROPERTIES OF RPC

3.1 Introduction

This chapter presents an experimental program to investigate the mechanical properties of the Reactive Powder Concrete (RPC) used in this study. The RPC was designed to have a nominal compressive strength of 100 MPa due to the limitation of the loading capacity of the testing machine. The experimental program included in this chapter consists of several tests to determine the compressive strength, compressive stress-strain relationship, flexural strength, tensile strength and tensile stress-strain relationship at the age of 28 days. The tensile strength of the RPC was the main focus among the other mechanical properties. Different test procedures were used to determine the tensile strength of the RPC. An experimental evaluation for these test procedures is also presented in this chapter. The experimental program was performed in the High Bay Laboratory of the School of Civil, Mining and Environmental Engineering at the University of Wollongong, Australia.

3.2 Experimental program

The experimental program of this study consisted of 48 concrete cylinder specimens that were cast and tested to determine the compressive strength, compressive stress-strain relationship, splitting tensile strength and double punch tensile strength of the RPC. In addition, 24 concrete prism specimens were also cast and tested to determine the direct tensile strength and the flexural strength of the RPC.
3.2.1 Materials

As mentioned earlier, the RPC was designed to have a nominal compressive strength of 100 MPa due to the limitation of the loading capacity of the testing machine. To achieve the targeted compressive strength of the RPC in this study, several trail mixes were conducted as shown in Appendix B. Four RPC mixes were produced with general purpose cement 800 kg/m³, fine sand 1050 kg/m³, densified silica fume 250 kg/m³, water 180 kg/m³ and superplasticizer 60 kg/m³. Steel fibres were added by weight of 0 kg/m³, 80 kg/m³, 160 kg/m³ and 240 kg/m³ for 0%, 1%, 2% and 3% by volume of the RPC, respectively. The superplasticizer Viscocrete 3015LF (2016) was used in this study and complied with the specifications of ASTM C494 (2015). The steel fibres were provided by Ganzhou Daye Metallic Fibres (2015), having the dimensions of 13 mm in length and 0.2 mm in diameter with a maximum tensile strength of 2500 MPa.

For the purposes of this study, each RPC mix was recognized with an acronym. Mixes RPC0, RPC1, RPC2 and RPC3 refer to RPC mixes reinforced with 0, 1, 2 and 3 volumetric percentage of steel fibre, respectively.

3.2.2 Mixing, casting and curing of specimens

An electronic balance was used to weigh all the dry materials that were mixed in a laboratory mixer of 0.1 m³ capacity. First, all dry materials (cement, fine sand and densified silica fume) were mixed together for 5 minutes. Then, the water and the superplasticizer were added to the dry mixture. After a period of 10 minutes of mixing, the full amount of steel fibres was added and the desired flowability (Flow table test >120 mm) was obtained in accordance with ASTM C230 (2014). The flow table test showed that the flowability of RPC mixes decreased with the increase of steel fibre
percentage. The results of flow table test were 220 mm, 205 mm, 180 mm and 145 mm for Mixes RPC0, RPC1, RPC2 and RPC3, respectively.

The fresh RPC was then placed into the moulds of the specimens. During the process of concrete placing, an electric vibrator was used to compact and eliminate air voids. Plastic sheets were used for covering the specimens for a period of 24 hours. After that, the concrete was set and all specimens were taken out the moulds in order to start the curing process where the cylinders immersed into water tank of moisture curing for a period of 27 days. The purpose of the curing process is to provide the concrete cylinders with enough amount of water that enhance the hydration process of the cement to obtain the maximum strength of the concrete.

3.2.3 Test setup and procedure

3.2.3.1 Compressive test

To determine the compressive strength of the RPC at the age of 28 days, the Avery compression testing machine with loading capacity of 1800 kN was used. For each RPC mix, three cylinders were tested under constant loading rate of 20 MPa/min according to AS 1012.9 (1999). During the test, the Avery testing machine shows the reading of the applied load. As the concrete cylinder reached its maximum load, the load gauge stopped increasing and the reading of the load was manually recorded. Then, the compressive strength of the RPC can be simply calculated by dividing the maximum applied load on the known area of the RPC cylinder. The average of three samples was used to determine the compressive strength of each mix of the RPC.
3.2.3.2 Compressive stress-strain behaviour

The compressive stress-strain behaviour tests were carried out using the Denison universal testing machine with a loading capacity of 5000 kN, as shown in Figure 3.1. One Linear Variable Differential Transformer (LVDT) was used to measure the axial deformation of the mid-height region of 115 mm. In addition, two LVDTs were attached to the lower loading head of the machine to measure the total axial deformation of the specimens. All specimens were axially loaded with a displacement rate of 0.3 mm/minute until the load resistance of the specimens dropped to 30% of the peak load. The LVDTs and the load cell were connected to a data logger to record the readings every two seconds.

Figure 3.1 Test setup for compression stress-strain test
3.2.3.3 Flexural test

The flexural strength test was conducted in accordance with AS 1012.11 (1985). The Avery testing machine with loading capacity of 300 kN was used to obtain the flexural strength of the RPC. Three prism specimens with a cross-section of 100 mm × 100 mm and a length of 500 were tested under four-point loading as shown in Figure 3.2. The load was applied without shock with a rate of loading of 1 MPa/min according to AS 1012.11 (1985). The Avery testing machine only shows the reading of the applied load. Once the concrete prism reached its maximum load, the load gauge stopped increasing and the reading of the load was manually recorded. The flexural strength was calculated using Equation 3.1, according to AS 1012.11 (1985):

\[
f_{cf} = \frac{PL(1000)}{BD^2}
\]  

(3.1)

where, \(f_{cf}\) is the flexural strength in MPa, \(P\) is the failure load in kN, \(L\) is the span length in mm, \(B\) is the average width of the prism at the point of failure in mm, \(D\) is the average depth of prism at the point of failure in mm.

Figure 3.2 Test setup for flexural test
3.2.3.4 splitting test

The splitting tests were conducted according to AS 1012.10 (2000). Three cylinders (150 mm diameter × 300 mm height) of each RPC mix were tested to determine the average splitting strength. Two timber strips having the dimensions of 400 mm in length, 25 mm in width and 5 mm in thickness were located between the loading heads of the Avery testing machine and the specimen as bearing strips, as shown in Figure 3.3. The Avery testing machine with loading capacity of 1800 kN was used to test the concrete cylinders at a loading rate of 1.5 MPa/min according to AS 1012.10 (2000). Equation 3.2 was used to calculate the splitting tensile strength according to AS 1012.10 (2000).

\[ T = \frac{2000P}{\pi LD} \]  

(3.2)

where \( T \) is the splitting tensile strength in MPa, \( P \) is the maximum applied load in kN, \( L \) is the length of the specimen in mm, and \( D \) is the diameter of the specimen in mm.

Figure 3.3 Test setup for splitting test
3.2.3.5 Double Punch Test

The test procedure of the Double Punch Test (DPT) in Chen (1970) was adopted to perform the DPT in this study. Three cylinders with a diameter of 150 mm and a height of 150 mm were tested to determine the average DPT tensile strength of each RPC mix, see Figure 3.4. The Avery testing machine with loading capacity of 1800 kN was used to test the concrete cylinders with a loading rate of 1.4 MPa/min. Two steel punches were used to transfer the load from the testing machine to the concrete specimen, as shown in Figure 3.4. Each cylindrical punch had a diameter of 37.5 mm and a height of 25 mm, according to Chen (1970). Equation 3.3, as suggested by Chen (1970), was used to calculate the DPT tensile strength:

\[
f_{DPT} = \frac{P}{\pi(0.6dh - 0.25x^2)}
\]  

(3.3)

where, \( f_{DPT} \) is the double punch tensile strength in MPa, \( P \) is the maximum applied load in kN, \( d \) is the diameter of the specimen in mm, \( h \) is the height of the specimen in mm, and \( x \) is the diameter of the steel punch in mm.

![Figure 3.4 Test setup for DPT](image)
3.2.3.6 Direct tensile test

Several test setups were used in the literature for testing the direct tensile strength of concrete. The test setup used in this study was proposed in Alhussainy et al. (2016). The direct tensile test was performed on RPC prism specimens with a cross-section of 100 mm × 100 mm and a length of 500 mm. A wooden formwork was used as a mould for the specimens, as shown in Figure 3.5. To apply the direct tensile force on the concrete specimen, two steel gripping claws were used at the ends of the specimen. These claws were made of a 20 mm diameter threaded steel bar and embedded in the specimen to 125 mm. Four steel pins with 8 mm diameter and 30 mm length were welded to the threaded bar at an angle of 90º at 20 mm spacing to provide adequate anchorage between the steel claw and the concrete.

![Figure 3.5 Formwork of DTT specimens](image)

The gripping claws were fixed to the wooden mould by a nut and a washer from the outside of the formwork and a washer from the inside of the formwork. The washer inside was welded to the threaded bar to ensure an accurate alignment of the claws, as
shown in Figure 3.6. In order to prompt the failure to occur in the middle of the specimen, the cross sectional area of the specimen was reduced by using a timber triangle at the middle from two sides.

![Figure 3.6 Dimensions of specimens for the DTT (Adopted from Alhussainy et al. 2016)](image)

To avoid any misalignment of the claw during the testing, two universal joints as shown in Figure 3.7 were designed by Alhussainy et al. (2016). The universal joints were used to grip the ends of the specimen by the testing machine to apply axial tensile forces to the specimen. As illustrated in Figure 3.8, the specimen aligned vertically between the jaws of the testing machine due to the free movement provided by the joints at the ends of the specimen.

![Figure 3.7 Universal joints](image)
Figure 3.8 Test setup for the DTT

In order to measure the concrete tensile strain during the test, one strain gauge (PL-90-11) with a gauge length of 90 mm was attached to the mid-length of the specimen. The PL-90-11 strain gauges were manufactured by Tokyo Sokki Kenkyujo Company, Tokyo, Japan (TML 2015). All specimens were axially loaded up to failure with a displacement controlled loading at 0.2 mm/min and the data (load, displacement and strain) were recorded at every two seconds. It is noted that the direct tensile strength was calculated as the maximum tensile load divided by the reduced cross sectional area of the specimens (100 mm × 80 mm).

3.2.4 Results and discussion

3.2.4.1 Compressive strength

Table 3.1 shows the mechanical properties of the RPC mixes at the age of 28 days. The compressive strength test of RPC was conducted according to AS 1012.9 (2014). The test results of the compressive strength are presented in Table 3.1. The average
Compressive strength of the four RPC mixes at the age of 28 days ranged from 73 MPa to 113 MPa. The highest compressive strength of the RPC was achieved with 3% of steel fibre content. Compared to Mix RPC0, the compressive strength of Mixes RPC1, RPC2 and RPC3 was increased by 8.4%, 43.6% and 53.5%, respectively.

Table 3.1 Mechanical Properties of the RPC mixes at the age of 28 days

<table>
<thead>
<tr>
<th>Mix Label</th>
<th>Compressive test (MPa)</th>
<th>Flexural test (MPa)</th>
<th>Splitting test (MPa)</th>
<th>DPT (MPa)</th>
<th>DTT (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RPC0</td>
<td>70.6</td>
<td>73.4</td>
<td>14.1</td>
<td>12.6</td>
<td>5.8</td>
</tr>
<tr>
<td>RPC1</td>
<td>76.6</td>
<td>79.6</td>
<td>15.3</td>
<td>14.9</td>
<td>9.5</td>
</tr>
<tr>
<td>RPC2</td>
<td>108.3</td>
<td>105.5</td>
<td>19.6</td>
<td>20.5</td>
<td>11.9</td>
</tr>
<tr>
<td>RPC3</td>
<td>116.1</td>
<td>112.7</td>
<td>21.4</td>
<td>22.2</td>
<td>17.3</td>
</tr>
</tbody>
</table>

3.2.4.2 Compressive stress-strain behaviour

Table 3.2 and Figure 3.9 show the typical test results of the compressive stress-strain curves of Mixes RPC0, RPC1, RPC2 and RPC3. Compared to Mix RPC0, the presence of steel fibres in Mixes RPC1, RPC2 and RPC3 have a marginal effect on the pre-cracking stress. Mix RPC0 showed a softening stress-strain response of nearly 10% of the maximum stress with a corresponding axial strain of 0.0033, as shown in Table 3.2.
This followed by a sudden drop of the compressive stress accompanied with the explosive failure mode. Whereas Mixes RPC1, RPC2 and RPC3 experienced a strain softening stress-strain behaviour in the post-cracking stress which extended to nearly 50% of the maximum stress due to the effect of interaction between the concrete matrix and the steel fibres. By using 1% of steel fibre in Mix RPC1, the axial strain at the maximum stress was nearly the same compared to Mix RPC0. By increasing steel fibre content to 2% and 3%, however, the axial strain at the maximum stress of Mixes RPC2 and RPC3 increased by 27% and 133%, respectively, compared to Mix RPC0, as can be seen in Table 3.2. The best ductile compressive stress-strain behaviour was achieved by Mix RPC3 which had the highest volume content of steel fibre, as shown in Figure 3.9.

Table 3.2 Test results of the compressive stress-strain curves

<table>
<thead>
<tr>
<th>Mix Label</th>
<th>Maximum tensile stress (MPa)</th>
<th>Corresponding strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Single</td>
<td>Average</td>
</tr>
<tr>
<td>RPC0</td>
<td>70.6</td>
<td>73.4</td>
</tr>
<tr>
<td></td>
<td>75.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>74.4</td>
<td></td>
</tr>
<tr>
<td>RPC1</td>
<td>76.6</td>
<td>79.6</td>
</tr>
<tr>
<td></td>
<td>81.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>80.9</td>
<td></td>
</tr>
<tr>
<td>RPC2</td>
<td>108.3</td>
<td>105.5</td>
</tr>
<tr>
<td></td>
<td>105.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>103.3</td>
<td></td>
</tr>
<tr>
<td>RPC3</td>
<td>116.1</td>
<td>112.7</td>
</tr>
<tr>
<td></td>
<td>109.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>112.5</td>
<td></td>
</tr>
</tbody>
</table>
3.2.4.3 Flexural strength

Table 3.1 shows the average flexural strength results of Mixes RPC0, RPC1, RPC2 and RPC3. The test results show that the flexural strength was increased by the increase of volume fraction of steel fibres within the mix of RPC. Compared to Mix RPC0, the flexural strength of Mixes RPC1, RPC2 and RPC3 were increased by 18%, 62% and 76%, respectively. It can be seen from these results that the flexural strength of the RPC was improved more than the compressive strength by increasing the steel fibre content from 0% to 3% by volume of concrete.

3.2.4.4 Splitting strength

The typical failure modes of Mixes RPC0, RPC1, RPC2 and RPC3 are shown in Figure 3.10. In Figure 3.10a, Mix RPC0 showed one failure surface at the centre of the cylinder and along the line of the loading strip. Mix RPC0 experienced a sudden and brittle failure mode and the specimens have been completely split into two halves after the test.
However, for Mixes RPC1, RPC2 and RPC3, the failure was not brittle and the specimens remained nearly intact after the failure. The incomplete splitting failure of Mixes RPC1, RPC2 and RPC3 was because the steel fibres distributed the applied stresses through the failure surface. In addition, a compressive zone can be seen under the bearing bar which unevenly distributed the load along the direction of the load due to the effect of the steel fibres. Table 3.1 and Figure 3.11 present the test results of the average tensile strength of Mixes RPC0, RPC1, RPC2 and RPC3. The average splitting tensile strength of Mixes RPC1, RPC2 and RPC3 was increased by 47%, 108% and 180%, respectively, compared to Mix RPC0. The highest splitting tensile strength (17.4 MPa) was achieved by Mix RPC3, which had the highest compressive strength and 3% of steel fibre by volume of the RPC.

Figure 3.10 Typical failure mode of RPC specimens under the splitting test: (a) RPC0, (b) RPC1, (c) RPC2 and (d) RPC3
3.2.4.5 Double punch strength

Figure 3.12 shows the typical failure modes of Mixes RPC0, RPC1, RPC2 and RPC3 tested under DPT. Mixes RPC0, RPC1 and RPC2 failed in four radial cracks which have been reported as an ideal failure mode (Chen 1970; Chen and Yuan 1980; Marti 1989; Molins et al. 2009; Carmona et al. 2013). Mix RPC3, however, failed in five radial cracks due to the increase of steel fibre volume fraction, as shown in Figure 3.12d. The typical failure mode of Mix RPC0 is presented in Figure 3.12a. Four radial failure surfaces were observed at an angle of nearly 30° between each two close failure surfaces. By increasing the percentage of the steel fibre into the concrete mixture, the failure surfaces were observed at an equal angle of nearly 90°, as shown in Figure 3.12b, 3.12c and 3.12d. This behaviour could be due to the effect of steel fibre that distributes the stress in the RPC specimen during the test.
Figure 3.12 Typical failure mode of RPC specimens tested under the Double Punch Test (DPT): (a) RPC0, (b) RPC1, (c) RPC2 and (d) RPC3

Table 3.1 and Figure 3.11 show the test results of the DPT of all RPC mixes. The average tensile strength of Mixes RPC1, RPC2 and RPC3 increased by 26%, 65% and 106%, respectively, compared to Mix RPC0. Mix RPC3 was achieved the highest DPT tensile strength of 10.2 MPa, where the highest content of steel fibre was used. The results presented in Figure 3.11 indicate that the DPT is capable to detect variations in the steel fibre content of the RPC specimens.

3.2.4.6 Direct tensile strength

The typical failure modes of the DTT for Mixes RPC0, RPC1, RPC2 and RPC3 are shown in Figure 3.13. For Mixes RPC0 and RPC1 tested under direct tensile load, only one failure crack surface was observed at the middle of the specimens, as shown in Figures 3.13a, b. Different failure modes, however, were observed in Figures 3.13c, d where two and three failure crack surfaces were seen for Mixes RPC2 and RPC3,
respectively. No claw slippage was observed at the ends of all specimens, which indicated that adequate alignment was provided to the specimens under the DTT.

Table 3.1 and Figure 3.11 present the test results of Mixes RPC0, RPC1, RPC2 and RPC3 test under DTT. The minimum tensile strength of 4.5 MPa was obtained by Mix RPC0 and the maximum tensile strength value of 9.8 MPa was achieved by Mix RPC3 which has 3% of steel fibre by volume of RPC. The test results also show that the average direct tensile strength of Mixes RPC1, RPC2 and RPC3 increased by 30%, 74% and 120%, respectively, compared to RPC0.

Figure 3.13 Typical failure mode of RPC specimens tested under the DTT: (a) RPC0, (b) RPC1, (c) RPC2 and (d) RPC3

3.2.4.7 Comparison of tensile test methods

Table 3.3 and Figure 3.14 compare the results of the tensile strength of different test methods. In comparison to the tensile strength results of the DTT, figure 3.14 shows that the splitting test overestimates the tensile strength of the RPC. In addition, by increasing the steel fibre content, the overestimation of the tensile strength was increased. Table 3.3, shows that the splitting tensile strength of Mixes RPC0, RPC1, RPC2 and RPC3 was 39%, 57%, 66% and 77% higher than the direct tensile strength, respectively. This is due to the ductile behaviour of the RPC with steel fibre that
composes a wide compressive area under the bearing bar during the test, as can be seen in Figure 3.10b, 3.10c and 3.10d. Also, the value of the splitting tensile strength is calculated using Equation 3.2 assuming that the concrete specimen splits into two halves by one primary surface failure along the vertical diameter of the specimen. By introducing steel fibre to the RPC mixes, however, the horizontal tensile stress distributes along one primary surface failure and more than one secondary surface failure which creates a vertical failure zone instead of a surface failure, as can be seen in Figure 3.10. Thus, a higher result of tensile strength can be expected than the actual one.

Table 3.3 Tensile strengths of RPC obtained from different test methods

<table>
<thead>
<tr>
<th>Mix Label</th>
<th>Splitting test (ST) (MPa)</th>
<th>Double Punch Test (DPT) (MPa)</th>
<th>Direct tensile test (DTT) (MPa)</th>
<th>ST/DTT</th>
<th>DPT/DTT</th>
</tr>
</thead>
<tbody>
<tr>
<td>RPC0</td>
<td>6.2</td>
<td>4.97</td>
<td>4.46</td>
<td>1.39</td>
<td>1.11</td>
</tr>
<tr>
<td>RPC1</td>
<td>9.1</td>
<td>6.29</td>
<td>5.78</td>
<td>1.57</td>
<td>1.09</td>
</tr>
<tr>
<td>RPC2</td>
<td>12.9</td>
<td>8.21</td>
<td>7.78</td>
<td>1.66</td>
<td>1.06</td>
</tr>
<tr>
<td>RPC3</td>
<td>17.4</td>
<td>10.23</td>
<td>9.81</td>
<td>1.77</td>
<td>1.04</td>
</tr>
</tbody>
</table>

According to the results shown in Figure 3.14, the tensile strengths of the DPT were close to those obtained from the DTT. The DPT tensile strength of Mixes RPC0, RPC1, RPC2 and RPC3 was within 11% higher than the corresponding direct tensile strength, as shown in Table 3.3. Chen (1970) reported that the precision of the DPT was enhanced as the number of radial cracks increased. The higher the number of failure surfaces, the more uniform distribution of the stresses in the specimen occurs. Using of steel fibres within the RPC mixes can also result in more uniform distribution of the
stress in the specimen under DPT, as can be seen in Figure 3.12d. Based on the results discussed above, the DPT showed more accurate tensile strength than the splitting test when compared with the DTT for the RPC.

Figure 3.14 Comparison between the tensile strength of different test methods

3.2.4.8 Tensile stress-strain behaviour

The typical tensile stress-strain curves of all RPC mixes are shown in Figure 3.15. Table 3.4, also shows the test results of the ultimate tensile stress and the corresponding strain of specimens under DTT. For all RPC mixes, linear axial stress-strain behaviour up to the maximum stress was observed. As can be seen in Figure 3.15, the axial stress dropped to zero immediately after reaching the maximum stress in Mix RPC0. As expected, only one major crack was observed at the mid-length of Mix RPC0, see Figure 3.13a. The post-peak behaviour, however, changed by including 1%, 2% and 3% steel fibre by volume of the RPC.
Table 3.4 Test results of the DTT

<table>
<thead>
<tr>
<th>Mix Label</th>
<th>Maximum tensile stress (MPa)</th>
<th>Corresponding strain (%)</th>
<th>Maximum Tensile Load (kN)</th>
<th>Corresponding elongation (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Single</td>
<td>Average</td>
<td>Single</td>
<td>Average</td>
</tr>
<tr>
<td>RPC0</td>
<td>4.19</td>
<td>4.46</td>
<td>0.089</td>
<td>0.095</td>
</tr>
<tr>
<td></td>
<td>4.40</td>
<td></td>
<td>0.095</td>
<td>0.095</td>
</tr>
<tr>
<td></td>
<td>4.79</td>
<td></td>
<td>0.103</td>
<td></td>
</tr>
<tr>
<td>RPC1</td>
<td>5.52</td>
<td>5.78</td>
<td>0.109</td>
<td>0.116</td>
</tr>
<tr>
<td></td>
<td>5.84</td>
<td></td>
<td>0.117</td>
<td>0.116</td>
</tr>
<tr>
<td></td>
<td>5.98</td>
<td></td>
<td>0.122</td>
<td></td>
</tr>
<tr>
<td>RPC2</td>
<td>7.58</td>
<td>7.78</td>
<td>0.141</td>
<td>0.144</td>
</tr>
<tr>
<td></td>
<td>7.91</td>
<td></td>
<td>0.146</td>
<td>0.144</td>
</tr>
<tr>
<td></td>
<td>7.85</td>
<td></td>
<td>0.146</td>
<td></td>
</tr>
<tr>
<td>RPC3</td>
<td>9.70</td>
<td>9.81</td>
<td>0.203</td>
<td>0.209</td>
</tr>
<tr>
<td></td>
<td>9.96</td>
<td></td>
<td>0.218</td>
<td>0.209</td>
</tr>
<tr>
<td></td>
<td>9.77</td>
<td></td>
<td>0.207</td>
<td></td>
</tr>
</tbody>
</table>

For Mix RPC1, the axial tensile stress dropped to nearly one-third of the maximum load followed by a descending axial stress-strain curve. Mix RPC1 also failed with one major crack located in the middle of the specimen, as shown in Figure 3.13b. By increasing the steel fibre to 2% in Mix RPC2, the post-peak stress-strain curve was experienced a softening behaviour but without a sudden drop in the axial tensile stress. Two major cracks were observed in the failure mode of Mix RPC2, as shown in Figure 3.13c. For
Mix RPC3, however, the post-peak stress-strain curve showed a tensile strain hardening behaviour with three peaks of tensile stress where Mix RPC3 failed with three major cracks, as illustrated in Figure 3.13d. The maximum axial tensile stress of the RPC specimens increased due to the influence of an increase in the content of steel fibre, as can be seen in Figure 3.15. Thus, DTT results showed that the tensile strength of the RPC can be enhanced by increasing the content of steel fibres in the RPC mix and the tensile strain hardening can be achieved with 3% of steel fibre by volume of RPC.

Figure 3.15 Typical axial tensile stress-strain curves of RPC mixes

3.2.4.9 Relationship between tensile strength and compressive strength

The tensile strength $f_t$ is an important material property in the structural design. Most of the international design codes present an equation to predict the value of the tensile strength from the compressive strength $f'_c$. The ratio between these two parameters is affected by the type and strength of concrete. Several studies were conducted to present a simple and accurate model to predict $f_t$ of different types of concrete (Zain et al 2002;
ACI 363R-92 1992; CEB-FIB 1991; Arioglu et al. 2006; Ashour and Faisal 1993). In this study, some of the existing models were used to predict the $f_t$ of the four RPC mixes. Only models that cover a range of $f'_c$ from 70 MPa to 120 MPa were selected, as presented in Table 3.5.

Table 3.5 Existing equations to predict the tensile strength based on the compressive strength

<table>
<thead>
<tr>
<th>Equation No.</th>
<th>Model</th>
<th>Source</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>(3.4)</td>
<td>$f_t = \frac{f'_c}{0.1f'_c + 7.11}$</td>
<td>Zain et al. (2002)</td>
<td>High performance concrete, $f'_c &gt; 40$ MPa</td>
</tr>
<tr>
<td>(3.5)</td>
<td>$f_t = 0.59f'_c^{0.5}$</td>
<td>ACI 363R-92 (1992)</td>
<td>21 MPa $&lt; f'_c &lt; 83$ MPa</td>
</tr>
<tr>
<td>(3.6)</td>
<td>$f_t = 0.3f'_c^{2/3}$</td>
<td>CEB-FIB (1991)</td>
<td></td>
</tr>
<tr>
<td>(3.7)</td>
<td>$f_t = 0.321f'_c^{0.66}$</td>
<td>Arioglu et al. (2006)</td>
<td>15 MPa $&lt; f'_c &lt; 120$ MPa</td>
</tr>
<tr>
<td>(3.8)</td>
<td>$f_t = \frac{f'_c}{20 - \sqrt{FRI}} + 0.7 + \sqrt{FRI}$</td>
<td>Ashour and Faisal (1993)</td>
<td>Steel fibre reinforced concrete, FRI* is the fibre reinforcement index</td>
</tr>
<tr>
<td>(3.9)</td>
<td>$f_t = 0.21f'_c^{0.81}$</td>
<td>Xu and Shi (2009)</td>
<td>Steel fibre reinforced concrete</td>
</tr>
</tbody>
</table>

$*FRI = V_f \times \frac{l}{d}$ where, $FRI$ is the fibre reinforcement index, $V_f$ is the volume fraction of fibre, $l$ is the length of fibre and $d$ is the diameter of fibre.

To evaluate the predicted results of $f_t$ for the RPC, the slope of regression line between the experimental and the predicted results, the correlation factor ($R^2$) and the Average Absolute error (AAE) were used in this study, as can be seen in Table 3.5. The AAE was calculated according to Equation (3.10).

$$AAE = \frac{\sum_{i=1}^{N}[|p_{pre_i} - exp_i|]}{N} \quad (2.10)$$

According to the results illustrated in Table 3.5, all the values of the slope of regression line were $< 1$ which means all the selected models are conservative. The results also
showed that the predicted values of $f_t$ were closer to the experimental $f_t$ results of the RPC for the DTT and the DPT than the $f_t$ results of the splitting test, as presented in Table 3.5. Ashour and Faisal (1993) proposed Equation 3.8 for steel fibre reinforced concrete and they included the effect of the steel fibre (FRI) in this equation. Equation 2.8 obtained the highest values of slope of regression line and correlation factor between the experimental and predicted values of the $f_t$ compared to other equations. Equation 2.8 also obtained the lowest value of AAE of 40%, 13% and 7% for the splitting test, the DPT and the DTT, respectively. For this reasons, it can be concluded that Equation 2.8 yielded the most accurate prediction of $f_t$ among other equations.

Table 3.5 Validation of existing equations to predict the tensile strength of the RPC

<table>
<thead>
<tr>
<th>Equation No.</th>
<th>Source</th>
<th>Slope of regression line</th>
<th>$R^2$</th>
<th>AAE %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>ST DPT DTT</td>
<td>ST DPT DTT</td>
<td>ST DPT DTT</td>
</tr>
<tr>
<td>(3.4)</td>
<td>Zain et al. (2002)</td>
<td>0.10 0.210 0.210</td>
<td>0.860 0.892 0.920</td>
<td>44 21 21</td>
</tr>
<tr>
<td>(3.5)</td>
<td>ACI 363R-92 (1992)</td>
<td>0.10 0.230 0.250</td>
<td>0.865 0.897 0.922</td>
<td>44 21 20</td>
</tr>
<tr>
<td>(3.6)</td>
<td>CEB-FIB (1991)</td>
<td>0.161 0.357 0.384</td>
<td>0.866 0.898 0.920</td>
<td>41 17 16</td>
</tr>
<tr>
<td>(3.7)</td>
<td>Arioglu et al. (2006)</td>
<td>0.133 0.217 0.290</td>
<td>0.866 0.898 0.922</td>
<td>41 16 15</td>
</tr>
<tr>
<td>(3.8)</td>
<td>Ashour and Faisal (1993)</td>
<td>0.330 0.675 0.710</td>
<td>0.895 0.922 0.948</td>
<td>40 13 7</td>
</tr>
<tr>
<td>(3.9)</td>
<td>Xu and Shi (2009)</td>
<td>0.215 0.408 0.519</td>
<td>0.882 0.903 0.931</td>
<td>40 15 9</td>
</tr>
</tbody>
</table>
3.3 Summary

To summarize this chapter, it was found that there are many research studies carried out on the production of RPC which is an ultra-high performance concrete, but few studies have been published on the structural behaviour of RPC members, especially FRP confined RPC columns. According to the literature, RPC can be prepared and mixed in laboratory apparatus with acceptable mechanical properties. In terms of structural behaviour, the RPC without fibre is very brittle material and to increase the ductility of the RPC, steel fibres must be added. The ductility of RPC structural members such as columns can be significantly enhanced by using FRP confinement. More research studies are required to investigate the behaviour of FRP confined RPC columns.

In addition, the mechanical properties of the RPC were investigated in this chapter. Three different test methods were evaluated experimentally to determine the tensile strength of the RPC. According to the results shown above, the following conclusions can be drawn from this investigation:

- As expected, the best ductile compressive stress-strain behaviour of RPC mix was achieved with steel fibre of volume fraction of 3% which had the highest volume content of steel fibre.
- For the RPC, the splitting test was overestimating the tensile strength. In addition, by increasing the steel fibre content, the overestimation of the tensile strength was increased.
- The DPT showed more accurate tensile strength of the RPC than the splitting test when compared with the DTT.
- For the RPC mixes with steel fibre of volume fraction of 0%, 1%, 2% and 3%, the DPT was capable to detect the tensile strength of the RPC within the range of 11% higher than the direct tensile strength.

- Taking into account the low cost and the easy performance of the DPT, this test can be considered as an alternative to the DTT to obtain the tensile strength of the RPC.

- The tensile strength of the RPC can be enhanced by increasing the volume fraction of the steel fibres within the RPC mix and the tensile strain hardening can be achieved with 3% of steel fibre by volume of the RPC.

- The existing models that can be used to predict the tensile strength of the RPC yield more accurate results for the DPT and the DTT than the splitting test. However, more research is needed to develop a model that can precisely predict the tensile strength of the RPC.

The following chapter explains the stress-strain behaviour of FRP-confined hollow core specimens. In addition, hollow concrete specimens with and without PVC tube inside, having two different levels of FRP confinement.
4 EXPERIMENTAL STUDY ON THE BEHAVIOUR OF FRP-CONFINED HOLLOW CONCRETE COLUMNS WITH INTERNAL PVC TUBE

4.1 Introduction

In order to investigate the stress-strain behaviour of FRP-confined hollow specimens with a PVC tube inside, an experimental program has been performed for this purpose. This section shows in details the materials, fabrication and testing methods of all specimens that were used in this preliminary study. The effects of CFRP confinement, the hollow core and the internal PVC tube on the stress-strain behaviour of hollow concrete specimens are also presented in this preliminary study. The experimental work of this study was conducted in the high bay laboratory of the School of Civil, Mining and Environmental Engineering, University of Wollongong, Australia.

4.2 Aim and objectives

The main aim of this chapter is to investigate the stress-strain behaviour of CFRP confined hollow concrete specimens with an inner PVC tube under pure axial compression. For this aim, two degrees of CFRP confinement were used. The main objectives are as follows:

- Provide a comparison between the stress-strain behaviour of hollow concrete column and solid column confined with CFRP composites.
- Examine the stress-strain behaviour of CFRP confined hollow concrete specimens with and without inner PVC tube.
- Investigate the effects of two parameters: material strengths and specimens’ geometry on the behaviour of CFRP confined hollow concrete columns.
• Investigate the ductility of confined concrete columns with different configurations.

4.3 Experimental program

4.3.1 Configuration of specimens

In total, 18 cylinder specimens were prepared and tested under concentric axial load. All the concrete specimens had an outer diameter of 150 mm and a height of 300 mm. These specimens were divided into three groups according to the section configuration. Specimens in the first group consisted of six hollow core cylinders with an inner PVC tube (Figure 4.1a). The PVC tube had an outer diameter of 90 mm and a thickness of 1.5 mm. Specimens in the second group consisted of six hollow core cylinders with a central hole diameter of 90 mm (Figure 4.1b), while the six specimens in the third group were solid cylinders (Figure 4.1c). Each group consisted of one pair of control specimens without FRP confinement, one pair of FRP-confined specimens wrapped with one layer of carbon FRP (CFRP) sheet and the last pair of FRP-confined specimens were wrapped with two layers of CFRP sheet. The details of all the specimens are summarized in Table 4.1.

Each specimen is identified with an acronym (Table 4.1), which starts with a letter “HC” to represent hollow core specimens or “S” to represent solid specimens. For FRP-confined specimens, this is then followed by “1F” or “2F” to represent one or two layers of CFRP sheet. The letter “T” for some specimens is used to indicate that the specimens had an inner PVC tube. The number “1” or “2” at the end is used to differentiate two nominally identical specimens. For example, Specimen HCT-1F-1 was the first of two nominally identical hollow core specimens with an inner PVC tube and a one-layer CFRP wrap.
Table 4.1 Details of test specimens

<table>
<thead>
<tr>
<th>Specimen Type</th>
<th>Specimen Label</th>
<th>Inner Hole Diameter (mm)</th>
<th>Number of CFRP Layers</th>
<th>Inner PVC tube thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hollow core with inner PVC tube</td>
<td>HCT-1</td>
<td>87</td>
<td>----</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>HCT-2</td>
<td>87</td>
<td>----</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>HCT-1F-1</td>
<td>87</td>
<td>1</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>HCT-1F-2</td>
<td>87</td>
<td>1</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>HCT-2F-1</td>
<td>87</td>
<td>2</td>
<td>1.5</td>
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<td></td>
<td>HCT-2F-2</td>
<td>87</td>
<td>2</td>
<td>1.5</td>
</tr>
<tr>
<td>Hollow core</td>
<td>HC-1</td>
<td>90</td>
<td>----</td>
<td>----</td>
</tr>
<tr>
<td></td>
<td>HC-2</td>
<td>90</td>
<td>----</td>
<td>----</td>
</tr>
<tr>
<td></td>
<td>HC-1F-1</td>
<td>90</td>
<td>1</td>
<td>----</td>
</tr>
<tr>
<td></td>
<td>HC-1F-2</td>
<td>90</td>
<td>1</td>
<td>----</td>
</tr>
<tr>
<td></td>
<td>HC-2F-1</td>
<td>90</td>
<td>2</td>
<td>----</td>
</tr>
<tr>
<td></td>
<td>HC-2F-2</td>
<td>90</td>
<td>2</td>
<td>----</td>
</tr>
<tr>
<td>Solid</td>
<td>S-1</td>
<td>Solid</td>
<td>----</td>
<td>----</td>
</tr>
<tr>
<td></td>
<td>S-2</td>
<td>Solid</td>
<td>----</td>
<td>----</td>
</tr>
<tr>
<td></td>
<td>S-1F-1</td>
<td>Solid</td>
<td>1</td>
<td>----</td>
</tr>
<tr>
<td></td>
<td>S-1F-2</td>
<td>Solid</td>
<td>1</td>
<td>----</td>
</tr>
<tr>
<td></td>
<td>S-2F-1</td>
<td>Solid</td>
<td>2</td>
<td>----</td>
</tr>
<tr>
<td></td>
<td>S-2F-2</td>
<td>Solid</td>
<td>2</td>
<td>----</td>
</tr>
</tbody>
</table>
Figure 4.1 Details of test specimens

(a) Hollow core concrete specimens with an inner PVC tube

(b) Hollow core concrete specimens

(c) Solid concrete specimen
4.3.2 Preparation of formwork

Eighteen standard cylindrical steel moulds (150 mm in diameter by 300 mm in height) were used in this experimental program, as shown in Figure 4.2. In order to obtain the inner hollow core within these specimens, PVC tubes were used for this purpose. The most critical step in the formwork preparation is keeping the PVC tube in the centre of the specimens’ mould. For this aim, four steel pins have been used with a length of 150 mm ± 1 mm. These pins were located into two levels along the PVC tube in a cross shape, as shown in Figure 4.3 to provide more resistance against the horizontal movement.

![Figure 4.2 Steel moulds for concrete specimens](image)

The PVC tubes were prepared in 400 mm in length which extended 100 mm more than specimens’ length to provide bracing mechanism against vertical movement, as shown above in Figure 4.3. The PVC tubes were restrained against vertical movement by using steel wire on the extend part of the PVC tube. The purpose of this vertical restrain is to prevent flowing of fresh concrete inside the PVC tube from the bottom base during the
cast of the concrete, see Figure 4.3. The extended part of the tube was cut prior to the test of the specimens. For hollow core specimens without PVC tube (Specimens HC, HC-1F and HC-2F), the PVC tube was taken out the concrete specimen prior to the test of the specimens.

![Figure 4.3 Fixing the PVC tube at the centre of steel mould](image)

**4.3.3 Materials**

Normal strength concrete, PVC tube and CFRP sheet are the basic materials which have been used throughout the experimental program. All these materials were tested in the High Bay laboratory to investigate the desired mechanical properties.

**4.3.3.1 Concrete**

The concrete was designed to obtain a nominal compressive strength of 40 MPa at the age of 28 days. The concrete was made with commercially available materials: Bastion
Type 1 General Purpose cement that complied with the AS3972 (1997), Sydney aggregate with a range of particle size from 150 µm to 10 mm and fine grade Fly ash manufactured by Flyash Australia (Eraring Power Station). The superplasticizer Pozzolith 370PC produced by BASF Australia which complied with ASTM C494 (2015) was used in order to maintain the required workability of the concrete. Table 4.2 shows the mix design of the concrete that was used to cast all specimens.

Table 4.2 Concrete mix proportion of 40 MPa

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight for 1 m$^3$ (kg)</th>
<th>Weight for 0.1 m$^3$ (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>260</td>
<td>26 kg</td>
</tr>
<tr>
<td>Fly ash</td>
<td>100</td>
<td>10 kg</td>
</tr>
<tr>
<td>Fine sand</td>
<td>228</td>
<td>22.8 kg</td>
</tr>
<tr>
<td>Coarse sand</td>
<td>532</td>
<td>53.2 kg</td>
</tr>
<tr>
<td>10 mm aggregate</td>
<td>950</td>
<td>95.0 kg</td>
</tr>
<tr>
<td>Water</td>
<td>187</td>
<td>18.7 kg</td>
</tr>
<tr>
<td>Superplasticizer (Pozzolith 370PC)</td>
<td>(350 ml/100 kg of binder)</td>
<td>126 ml</td>
</tr>
</tbody>
</table>

Nine concrete cylinders (100 mm in diameter × 200 mm in length) were cast from the concrete batch. Before testing, the concrete cylinders were capped with high strength plaster paste (plaster to water ratio of 3.5:1) in order to prevent the premature failure of the concrete cylinders. The Avery compression testing machine was used to test the concrete cylinders under a constant loading rate of 20 MPa/min according to AS 1012.9 (1999). During the test, the Avery testing machine shows the reading of the applied load. Once the concrete cylinder reached its maximum load, the load gauge stopped increasing and the reading of the load was manually recorded. Then, the strength of the
concrete can be simply calculated by dividing the maximum applied load on the known area of the concrete cylinder.

For each testing age, three cylinders were tested according to AS 1012.9 (1999) to determine the average compressive strength at the age of 7 days, 28 days and testing day of the specimens, as shown in Figure 4.4. The average compressive strength at the age of 28 days was 41.9 MPa as illustrated in Table 4.3.

Table 4.3 Test results of concrete strength

<table>
<thead>
<tr>
<th>Testing Day</th>
<th>Concrete compressive strength (MPa)</th>
<th>Individual test</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7 days</td>
<td>26.7</td>
<td>27.4</td>
<td>27.1</td>
</tr>
<tr>
<td>28 days</td>
<td>42.4</td>
<td>42.6</td>
<td>40.6</td>
</tr>
<tr>
<td>Testing day of specimens</td>
<td>54.1</td>
<td>53.4</td>
<td>53.2</td>
</tr>
</tbody>
</table>

Figure 4.4 Compressive test of concrete
4.3.3.2 CFRP sheet

Unidirectional CFRP sheets were used to confine the specimens in the hoop direction. These CFRP sheets were manufactured by SGL GROUP The Carbon Company based in Wiesbaden, Germany (SGL Group 2014). The reason behind choosing CFRP sheets rather than other types of FRP materials is because of its outstanding properties to resist severe conditions compared with other FRP types. This material was supplied in roll of 50 m length and 300 mm width to cover the full length of specimens.

In order to determine the tensile strength of the CFRP sheet, coupon tests were performed according to ASTM D7565 (2010). A length of 250 mm and width of 25 mm are the dimensions of coupon test samples. A gripping length of 56 mm at both ends of the CFRP coupon was used to apply the tensile load on the samples, as shown in Figure 4.5. One and two layers of CFRP material were used in this test. For two layers of CFRP material, the layers were glued to each other with adhesive material by wet lay-up method. For all CFRP samples, both ends of the CFRP coupon were capped by aluminium taps in order to grip the CFRP coupon by the testing machine. Figure 4.6 shows the tensile test setup of the CFRP coupons. The tensile properties of the CFRP coupons are presented in Table 4.4 and Figure 4.7. The average tensile force per unit width and the strain at rupture for one CFRP layer were 1557 N/mm and 0.0135, respectively. Whereas, the average tensile force per unit width and the strain at rupture for two CFRP layers were 1968 N/mm and 0.0175, respectively.
Figure 4.5 Dimensions of CFRP coupon

Figure 4.6 Tensile test setup for the CFRP coupons
### Table 4.4 Test results of CFRP coupons

<table>
<thead>
<tr>
<th>Coupon No.</th>
<th>No. of CFRP layers</th>
<th>Strain at rupture</th>
<th>Tensile force per unit width at rupture (N/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0.0138</td>
<td>1669</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>0.0140</td>
<td>1534</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>0.0127</td>
<td>1467</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td><strong>0.0135</strong></td>
<td><strong>1557</strong></td>
</tr>
<tr>
<td>4</td>
<td>2</td>
<td>0.0176</td>
<td>2018</td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>0.0162</td>
<td>1928</td>
</tr>
<tr>
<td>6</td>
<td>2</td>
<td>0.0188</td>
<td>1957</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td><strong>0.0175</strong></td>
<td><strong>1968</strong></td>
</tr>
</tbody>
</table>

Figure 4.7 Tensile force-strain relations of CFRP coupons
4.3.3.3 PVC tube

PVC tubes (Type U) of an outer diameter of 90 mm, an inner diameter of 87 mm and a length of 300 mm were used in this study. This product complies with the requirements of AS/NZS 1477 (2009). Three coupons having the dimensions of 165 mm in total length, 57 mm in test length and 13 mm in test width were taken from the longitudinal direction of the PVC tube to obtain the tensile stress-strain relationship of this material according to ASTM D638 (2014), as can be seen in Figure 4.8.

The typical tensile stress-strain curve obtained from these tests is shown in Figure 4.9, where the tensile strain was obtained from a clip-on extensometer attached to the specimen. The ultimate tensile stress, the ultimate tensile strain and the elastic modulus were found to be 44.47 MPa, 54% and 3.6 GPa, respectively.

In addition, two PVC tubes with a length of 300 mm were tested under axial compression to obtain the maximum load capacity of the PVC tube and the test results are shown in Figure 4.10. The average maximum axial load of the PVC tubes under compression was 22 kN.

![Figure 4.8 Dimensions of the PVC coupon](image-url)
Figure 4.9 Typical tensile stress-strain behaviour of the PVC coupon

Figure 4.10 Axial compressive load-deformation behaviour of PVC tube
4.3.4 Mixing and curing of Concrete

One concrete mix with a nominal compressive strength of 40 MPa was used to cast all specimens. Concrete ingredients were prepared and mixed in the High Bay laboratory. The mix proportion of this concrete mixture is shown in Table 4.2 above which complies with the requirements of AS 1379 (2010) and AS 3600 (2009). An electronic balance was used to weigh all the dry materials (cement, Fly ash and aggregate) that were mixed in a laboratory mixer of 0.1 m$^3$ capacity. Then, the water and superplasticizer were added gradually to the dry mixture. The total time of mixing was 10 minutes. The slump test was performed on the fresh concrete according to the requirements of AS 1012 (1999) to obtain the desired workability. The fresh concrete was then placed into the formwork, as shown in Figure 4.11. During the process of concrete placing, an electric vibrator was used to compact and eliminate air voids. Plastic sheets were used for curing and covering the specimens for a period of 24 hours. After that, the concrete was set and all specimens were taken out the moulds in order to start the curing process where the cylinders were immersed into water tank for moisture curing for a period of 27 days. The purpose of the curing process is to provide the concrete cylinders with enough amount of water that enhance the hydration process of the cement to obtain the maximum strength of the concrete.
4.3.5 CFRP wrapping of specimens

Concrete specimens as mentioned above in this chapter were divided into three groups; hollow specimens with inner PVC tube, hollow specimens and solid specimens. Each group was also divided into three categories in terms of CFRP confinement; without confinement, with one layer of CFRP confinement and with two layers of CFRP confinement. However, specimens without confinement were only wrapped with one layer (50 mm in width) of CFRP sheet at the ends of these specimens to avoid premature failure of concrete during the test.

Bonding CFRP sheets (300 mm in width) to the exterior face of the specimens was made with adhesive material and lay-up method. Preparation of adhesive was according to the instructions of the manufacturer. Then the adhesive mixture was applied to the
concrete surface and the CFRP sheet was wrapped the specimen in the hoop direction with 50 mm overlap. Applying of two layers of CFRP sheets was also required to spread the adhesive between the two layers to provide typical bonding.

4.3.6 Setup of the test

All compression tests were carried out using the Denison universal testing machine with a loading capacity of 5000 kN, as shown in Figure 4.12. One LVDT was used to measure the axial strain of the mid-height region of 115 mm. The LVDT was attached to the two-ring frame that complied with the requirements of AS 1012.17 (1997). In addition, two strain gauges with a gauge length of 5 mm were attached at the mid-height of the CFRP wrap to measure the hoop strains. The BFLA-5-8 strain gauges were manufactured by Tokyo Sokki Kenkyujo Company, Tokyo, Japan (TML 2015). Each specimen was loaded twice; the first loading was up to 5% of the estimated load carrying capacity which is basically for the seating of the gauges (LVDT and strain gauges). No readings were recorded during the first loading. Then, all specimens were axially loaded with a displacement rate of 0.5 mm/minute until the load resistance of the specimens dropped to 30% of the peak load. The LVDTs and the load cell were connected to a data logger to record the readings every two seconds.
4.3.7 Experimental results

4.3.7.1 Mode of Failure

All the eighteen concrete specimens were tested up to the ultimate load. In general, failure of specimens without CFRP sheets was noticed by crushing and spalling of concrete at the mid-height of the specimens. Failures of solid specimens were more likely to be accompanied with loud sound of sudden failure and it was not the case with hollow specimens. For specimens confined with CFRP sheets, snapping sounds were heard prior to the ultimate failure, revealing the rupture of CFRP composites and debonding between the layers of CFRP confinement and the concrete specimen. This type of failure was explosive but not sudden, and the CFRP composites failed due to the expanded concrete. As can be seen in Figure 4.13, concrete specimens without CFRP
confinement show brittle failure mechanism, while those confined with CFRP remained intact after failure. However, three out of the eighteenth specimens (S-1F-2, HC-1F-2 and HCT-1F-2) exhibited premature failure where concrete crushing occurred at one end of the specimen. For this reason, the results of those three specimens were neglected.

Figure 4.13 Modes of failure of the concrete specimens

4.3.7.2 Axial stress-strain behaviour

The key test results are summarized in Table 4.5. The axial stress-strain curves of concrete in solid specimens are compared with those of concrete in hollow specimens without a PVC tube in Figure 4.14. For clarity of presentation, the stress-strain curves of confined specimens are all terminated at a point corresponding to the rupture of CFRP.

Figure 4.14a shows that the unconfined strength of hollow specimens was slightly lower than that of solid specimens (see also Table 4.5). In addition, the hollow specimens generally had a steeper descending branch than the solid specimens, suggesting that the inner void had a negative effect on both the strength and ductility of the specimen.
Figures 4.14b and 4.14c shows the behaviour of CFRP-confined hollow specimens is quite different from that of the corresponding solid specimens. The latter generally had a bilinear stress-strain curve while the curves of the former typically had a descending branch. As a result, the CFRP-confined hollow specimens generally had a much lower peak stress than the corresponding solid specimens, although the ultimate axial strains of the former were comparable to or even larger than the latter. For hollow specimens, the stress decreased more rapidly after the peak value for specimens with a weaker CFRP wrap (see Figures 4.14b and 4.14c).

Figure 4.15 shows a comparison between the stress-strain curves of concrete in hollow specimens and those of concrete in the corresponding specimens with a PVC tube. When calculating the axial stress of concrete in the latter, the load contribution of the PVC tube was ignored as it was generally rather small (peak load = 22 kN) compared with that of the concrete (peak load of unconfined concrete = 522 kN).

Figure 4.15a shows that the presence of an inner PVC tube had a marginal effect on the behaviour of the unconfined concrete. Figure 4.15b, however, shows that the additional PVC tube changed the post-peak behaviour of the stress-strain curve reduced the of one-layer CFRP-confined specimens. For the specimen without PVC tube (i.e. HC-1F-1), the stress in the second branch was decreased by 33% compared to the peak stress, while for the specimen with a PVC tube (i.e. HCT-1F-1) the stress in the second branch was decreased by only 9% compared to the peak stress.

For two-layer CFRP-confined specimens, Figure 4.15c shows that the effect of PVC tube was even more obvious: Specimens HCT-2F-1, 2 had a bilinear stress-strain curve with two ascending branches. By contrast, the curves of the two specimens without a PVC tube both had a clear descending second branch which was lower than that of their
counterparts with a PVC tube. This is believed to be due to two important functions of the inner PVC tube: (1) preventing local spalling failure of concrete near the inner edge; and (2) providing inner pressure to the annular concrete section.

### Table 4.5 Key test results

<table>
<thead>
<tr>
<th>Specimen Label</th>
<th>Maximum Stress $f_{cc}$ (MPa)</th>
<th>Strain at Maximum Stress $ε_{max}$</th>
<th>Strain at Rupture of FRP $ε_{cc}$</th>
<th>Hoop Rupture Strain $ε_{h,rupt}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f_{cc}$</td>
<td>Average $f_{cc}$</td>
<td>Average $ε_{max}$</td>
<td>Average $ε_{cc}$</td>
</tr>
<tr>
<td>HCT-1</td>
<td>46.7</td>
<td>46.2</td>
<td>0.0028</td>
<td>---</td>
</tr>
<tr>
<td>HCT-2</td>
<td>45.7</td>
<td></td>
<td>0.0027</td>
<td>---</td>
</tr>
<tr>
<td>HCT-1F-1</td>
<td>54.1</td>
<td>54.1</td>
<td>0.0053</td>
<td>0.0053</td>
</tr>
<tr>
<td>HCT-2F-1</td>
<td>66.5</td>
<td>65.0</td>
<td>0.0281</td>
<td>0.0199</td>
</tr>
<tr>
<td>HCT-2F-2</td>
<td>63.5</td>
<td></td>
<td>0.0116</td>
<td>0.0293</td>
</tr>
<tr>
<td>HC-1</td>
<td>46.8</td>
<td>46.9</td>
<td>0.0023</td>
<td>0.0024</td>
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<td>46.9</td>
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<td>0.0025</td>
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</tr>
<tr>
<td>HC-1F-1</td>
<td>54.5</td>
<td>54.5</td>
<td>0.0048</td>
<td>0.0048</td>
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<tr>
<td>HC-2F-1</td>
<td>60.9</td>
<td>62.8</td>
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<td>0.0065</td>
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<td></td>
<td>0.007</td>
<td>0.0261</td>
</tr>
<tr>
<td>S-1</td>
<td>49.6</td>
<td>48.9</td>
<td>0.0028</td>
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</tr>
<tr>
<td>S-2</td>
<td>48.13</td>
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<td>0.0032</td>
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<td>S-1F-1</td>
<td>70.3</td>
<td>70.3</td>
<td>0.0316</td>
<td>0.0316</td>
</tr>
<tr>
<td>S-2F-1</td>
<td>104.6</td>
<td>101.8</td>
<td>0.0214</td>
<td>0.0233</td>
</tr>
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<td>S-2F-2</td>
<td>98.9</td>
<td></td>
<td>0.0251</td>
<td>0.0251</td>
</tr>
</tbody>
</table>
Figure 4.14 Axial stress-strain curves of solid and hollow specimens
Figure 4.15 Axial stress-strain curves of hollow with and without PVC tube specimens

(a) Unconfined Hollow and hollow with inner PVC tube specimens

(b) Hollow and hollow with inner PVC tube specimens confined with one layer of CFRP

(c) Hollow and hollow with inner PVC tube specimens confined with two layers of CFRP

Figure 4.15 Axial stress-strain curves of hollow with and without PVC tube specimens
While Figures 4.15b and 34.15c clearly show the beneficial effect of the inner PVC tube, it may be noted that such an effect does not seem to be significant. This was due to the use of a thin PVC tube in the present study whose stiffness was rather small. The PVC tube had a thickness of 1.5 mm and an elastic modulus of 3.6 GPa, so in terms of axial stiffness it was only equivalent to a steel tube of the same diameter and a thickness of around 0.03 mm. When a thicker PVC tube is used, it can be expected that the beneficial effect of the inner tube would be more pronounced.

4.3.7.3 Comparison between inner tube of PVC and steel

To compare between the behaviour of PVC tube and steel tube, the results of four FRP-confined DSTC specimens with inner steel tube (2.1 mm thickness) were selected from previous study that was conducted by Wong et al. (2008). The four FRP-confined DSTC specimens (D37-C1-I, D37-C1-II, D37-C2-I and D37-C1-II) were selected for the comparison among other specimens because they had relatively similar dimensions (152.5 mm diameter × 305 mm height × 88 mm inner void) to the specimens with inner PVC tubes presented in this study. In addition, these specimens had nearly the same strength of the FRP confinement (average tensile strength of 1825.5 MPa).

Table 4.6 shows the results of the axial stress and axial strain of the four DSTC specimens and Specimens HCT-1F-1, HCT-2F-1 and HCT-2F-2, in which $f_{cc}$ is the maximum axial stress, and $\varepsilon_{cc}$ is the ultimate strain at the rupture of FRP confinement.

The results presented in Table 4.6 showed that the axial stress increase ($f_{cc}/f_{c}^{'}$) of Specimens D37-C1-I, II was 10.7% higher than Specimen HC1FT-1. Also, the stress increase of Specimens D37-C2-I, II was 19.4% higher than Specimens HCT-2F-1, 2. Whereas, the strain increase ($\varepsilon_{cc}/\varepsilon_{co}$) of Specimens HC1FT-1 and HCT-2F-1, 2 were 71.9% and 17.3% higher than Specimens D37-C1-I, II and D37-C2-I, II, respectively.
Thus, using an internal steel tube in FRP-confined DSTC specimen has the advantage of increasing the axial stress capacity. The using of an internal PVC tube however, has the advantage of increasing the axial strain. In addition, the PVC tube has the advantages of low cost, low self-weight and easy of fabrication over the steel tube.

<table>
<thead>
<tr>
<th>Specimens Label</th>
<th>Maximum Stress $f_{cc}$ (MPa)</th>
<th>Stress enhancement</th>
<th>Strain at Rupture of FRP tube $\varepsilon_{cc}$</th>
<th>Strain enhancement $\varepsilon_{cc}/\varepsilon_{co}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f_{cc}$</td>
<td>Average $f_{cc}$</td>
<td>$f_{cc}/f'_c$</td>
<td>Average $\varepsilon_{cc}$</td>
</tr>
<tr>
<td>HCT-1F-1</td>
<td>54.1</td>
<td>54.1</td>
<td>1.03</td>
<td>0.0295</td>
</tr>
<tr>
<td>HCT-2F-1</td>
<td>66.5</td>
<td>65.0</td>
<td>1.24</td>
<td>0.0281</td>
</tr>
<tr>
<td>HCT-2F-2</td>
<td>63.5</td>
<td></td>
<td></td>
<td>0.0293</td>
</tr>
<tr>
<td>D37-C1-I</td>
<td>42.9</td>
<td>42.2</td>
<td>1.14</td>
<td>0.0166</td>
</tr>
<tr>
<td>D37-C1-II</td>
<td>41.4</td>
<td></td>
<td></td>
<td>0.0133</td>
</tr>
<tr>
<td>D37-C2-I</td>
<td>55.9</td>
<td>54.4</td>
<td>1.48</td>
<td>0.0235</td>
</tr>
<tr>
<td>D37-C2-II</td>
<td>52.9</td>
<td></td>
<td></td>
<td>0.0188</td>
</tr>
</tbody>
</table>

**4.3.7.4 Axial strain-hoop strain behaviour**

Figures 4.16 and 4.17 show the axial-hoop strain curves of one-layer and two-layer CFRP-confined specimens, respectively. In the two figures, the axial strains were obtained from readings of the LVDT while the hoop strains were averaged from two strain gauges attached at the mid-height of the CFRP wrap.

It is evident from both figures that the lateral expansion behaviour of hollow specimens was quite different from that of the corresponding solid specimens. Such difference became significant after an axial strain of around 0.0025, when the lateral expansion of concrete started to increase rapidly. In hollow columns, the concrete could move
towards the inner void because of the absence of constraints from inside, leading to a reduced outward expansion as measured by the hoop strain gauges on the outer CFRP wrap (Figures 4.16 and 4.17). It is easy to understand that the curves of the specimens with a PVC tube generally lie between those of the corresponding solid and hollow specimens, due to the inner constraint/confinement provided by the PVC tube.

![Figure 4.16 Axial-hoop strain response of one layer of CFRP confinement](image1)

![Figure 4.17 Axial-hoop strain responses of two layers of CFRP confinement](image2)

It should also be noted that the effect of PVC tube on the outward expansion of concrete appeared to be more obvious for two-layer specimens (Figure 4.17) than one-layer
specimens. This was probably due to the stronger confinement provided by the two-layer CFRP, which led to more significant inward movement of the inner surface and in turn activated the PVC tube more effectively. Figure 4.18 shows the shape of two PVC tubes after test in Specimens HCT-1F-1 and HCT-2F-1, respectively. It is evident that the deformation of the latter was much more significant than the former.

Figure 4.18 PVC tube deformations of Specimens HC1FT-1 and HC2FT-1

4.3.7.5 Effect of CFRP confinement on the stress-strain behaviour

The effect of CFRP confinement is illustrated in Figure 4.19. As expected, for solid specimens, both the strength and the ductility of concrete was much enhanced because of the confinement of CFRP, and such enhancement was more pronounced for specimens with a two-layer CFRP than those with a one-layer CFRP (Figure 4.19a). The post-peak behaviour of the stress-strain curve in FRP-confined concrete specimens highly depends on the amount and the properties of the FRP material. If sufficient FRP confinement \( f_{l,a}/f'_{co} \geq 0.07 \) is provided, then hardened bilinear stress-strain behaviour is expected with enhanced strength and strain being achieved, Lam and Teng (2003).
Figure 4.19 Effect of CFRP confinements on stress-strain response

(a) Solid specimens

(b) Hollow specimens

(c) Hollow specimens with inner PVC tube
Figure 4.19b shows the effect of CFRP confinement for hollow specimens, where all the curves had a descending branch. Again, the behaviour of the specimens depended on the amount of confining CFRP, and the two-layer specimens (i.e. HC-2F-1, 2) are shown to have the largest strength. It may be noted that the stress decrease in the descending branch became less when a stronger CFRP wrap was used.

Figure 4.19c shows the effect of CFRP confinement for hollow specimens with a PVC tube. The shape of the curves was found to be significantly affected by the FRP confinement: the curves of the two unconfined specimens (i.e. HCT-1, 2) both had a descending branch, the one-layer specimen (i.e. HCT-1F-1) had an approximately elastic-perfectly plastic curve, while the two-layer specimens (i.e. HCT-2F-1, 2) had a hardened bilinear curve. Besides, the use of a stiff FRP jacket, the superior performance of two-layer specimens was believed to be also partially due to the more effective confinement provided by the PVC tube from inside as discussed above.

4.3.7.6 Ductility of specimens

The ductility of concrete columns is considered as one of the structural design aspects that need to be taken into account, particularly when concrete columns are resisting a high axial load. Ductility can be improved by using CFRP sheets to confine concrete columns. In this study, the calculation of the ductility depends on the axial stress-strain behaviour of the confined concrete which is the main component taking axial loads. The calculation method used is according to GangaRao et al. (2007) which is suitable for concrete with softened and hardened axial stress-strain behaviour, as shown in Equation 4.1 below:
Figure 4.20 Definitions for yield stress and yield strain (a) drop after yield; (b) softening after yield; (c) hardening after yield.
\[ \mu_e = \frac{\varepsilon_u}{\varepsilon_y} \] \hspace{1cm} (4.1)

where \( \mu_e \) is the specimen’s ductility, \( \varepsilon_u \) is the specimen’s strain at 85% of the maximum stress at post-yielding (for unconfined specimens) or is equal to the specimen’s strain at rupture of FRP confinement, \( \varepsilon_{cc} \) (for FRP-confined specimens) and \( \varepsilon_y \) is the strain at yield stress.

The method of defining the yield point is based on the equivalent elasto-plastic method that was suggested by Park (1989). In this study, three different types of stress-strain curves were observed. Figure 4.20 shows how yield stresses and yield strains are determined.

The results of ductility in this study are summarized in Table 4.7. In general, test results indicate that the ductility of concrete specimens can be significantly improved by applying CFRP confinement. Applying double confinement of the CFRP layers shows an outstanding improvement in term of the specimens’ ductility. The highest average value of ductility 15.5 was achieved by Specimens HCT-2F (Hollow specimen with an inner PVC tube and two layers of CFRP confinement), while the lowest average value of ductility 2.25 was obtained by Specimens HC (unconfined hollow specimen).

Table 4.7 presents the ductility values of the specimens and shows the comparative results of the ductility of confined CFRP specimens and unconfined ones. According to the results presented in Table 4.7, the ductility of hollow specimens can be enhanced to be of equal values to those of the FRP-confined solid specimens by using PVC tube for internal confinement. Figure 4.21 presents a comparison between the normalized average maximum stress and the normalized average ductility for all specimens.
<table>
<thead>
<tr>
<th>Specimen Label</th>
<th>Strain ε</th>
<th>Ductility μ</th>
<th>Average Ductility</th>
<th>Normalized average ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Strain at Yield stress $f_y$</td>
<td>Strain at Rupture stress $\varepsilon_{cc}$</td>
<td>Strain at 85% of $f_{cc}$</td>
<td></td>
</tr>
<tr>
<td>HCT-1</td>
<td>0.0018</td>
<td>---</td>
<td>0.0046</td>
<td>2.5</td>
</tr>
<tr>
<td>HCT-2</td>
<td>0.0018</td>
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<td>0.0042</td>
<td>2.3</td>
</tr>
<tr>
<td>HCT-1F-1</td>
<td>0.0020</td>
<td>0.0295</td>
<td>----</td>
<td>14.8</td>
</tr>
<tr>
<td>HCT-2F-1</td>
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<td>0.0281</td>
<td>----</td>
<td>15.6</td>
</tr>
<tr>
<td>HCT-2F-2</td>
<td>0.0019</td>
<td>0.0293</td>
<td>----</td>
<td>15.4</td>
</tr>
<tr>
<td>HC-1</td>
<td>0.0016</td>
<td>---</td>
<td>0.0033</td>
<td>2.1</td>
</tr>
<tr>
<td>HC-2</td>
<td>0.0016</td>
<td>---</td>
<td>0.0038</td>
<td>2.4</td>
</tr>
<tr>
<td>HC-1F-1</td>
<td>0.0021</td>
<td>0.0267</td>
<td>----</td>
<td>12.7</td>
</tr>
<tr>
<td>HC-2F-1</td>
<td>0.0020</td>
<td>0.0274</td>
<td>----</td>
<td>13.7</td>
</tr>
<tr>
<td>HC-2F-2</td>
<td>0.0020</td>
<td>0.0261</td>
<td>----</td>
<td>13.1</td>
</tr>
<tr>
<td>S-1</td>
<td>0.0018</td>
<td>---</td>
<td>0.0056</td>
<td>3.3</td>
</tr>
<tr>
<td>S-2</td>
<td>0.0017</td>
<td>---</td>
<td>0.0048</td>
<td>3.0</td>
</tr>
<tr>
<td>S-1F-1</td>
<td>0.0019</td>
<td>0.0243</td>
<td>----</td>
<td>12.7</td>
</tr>
<tr>
<td>S-2F-1</td>
<td>0.0015</td>
<td>0.0214</td>
<td>----</td>
<td>14.2</td>
</tr>
<tr>
<td>S-2F-2</td>
<td>0.0015</td>
<td>0.0251</td>
<td>----</td>
<td>16.7</td>
</tr>
</tbody>
</table>
Summary

Using FRP materials in confinement of concrete columns has been excessively studied and it has been agreed that this type of confinement results in an increment in the load carrying capacity of the column in addition to ductility enhancement. The stress-strain behaviour of confined concrete columns is well understood through extensive studies. However, for hollow circular columns, there is a limited number of studies that involved in explaining the stress-strain behaviour of this kind of columns. Thus, this chapter has presented and interpreted the results of a series of compression tests on CFRP-confined hollow concrete specimens with and without an inner PVC tube. The failure mode, axial stress-strain behaviour and axial-hoop strain behaviour of the test specimens have been discussed. Based on the test results and discussions presented above, the following conclusion points can be drawn:

1. The inner void in a concrete cylinder led to a slight decrease in the strength and ductility of unconfined concrete.

2. CFRP-confined hollow specimens with an inner PVC tube generally possessed good ductility and were superior to their counterparts without a PVC tube. This
was due to the beneficial effect of the PVC tube which provided constraints/confinement from the inside.

3. Under the same axial strain, the outward lateral expansion of CFRP-confined hollow specimens was generally lower than the corresponding solid specimens. This suggests that the ultimate axial strain of the former may be larger than the latter for the same confining material.

4. Compared with hollow specimens without an inner tube, the presence of an inner PVC tube led to an increased outward expansion of the CFRP-confined specimens, but this effect was only obvious when the CFRP confinement was strong (i.e. by using a two-layer wrap).

5. For unconfined specimens, solid specimens exhibited higher ductility than hollow specimens. For confined specimens, however, the ductility of hollow specimens with an internal PVC tube can be enhanced to show equal values of ductility compared to those of the solid specimens.

It should also be noted that the PVC tube used in the present chapter had only a small stiffness. Further research is needed to investigate the effect of thickness of PVC tube. It can be expected the beneficial effects are even more pronounced than those presented in this study if a thicker PVC tube was used.

The next chapter (Chapter 5) explains the behaviour of steel reinforced hollow core Reactive Powder Concrete (RPC) circular specimens confined with a CFRP tube. A thicker PVC tube is used than the one used in this chapter. Chapter 5 also investigates the behaviour of the CFRP tube confined hollow core RPC specimens under different loading conditions concentric load, eccentric load of 25 mm and 50 mm and four-point bending.
5 EXPERIMENTAL STUDY ON THE BEHAVIOUR OF THE HCRPC COLUMNS

5.1 Introduction

This chapter presents an experimental program in order to evaluate the behaviour of steel reinforced hollow core Reactive Powder Concrete (HCRPC) circular specimens confined with a CFRP tube and to examine the efficiency of the CFRP tube in enhancing both the strength and ductility of this type of specimen. The CFRP tube confined HCRPC specimens were cast with and without internal tube. Two types of tubes were used as an internal tubes, PVC tube and steel tube. To investigate the behaviour of the CFRP tube confined HCRPC specimens under different loading conditions, the specimens were subjected to concentric load, eccentric load of 25 mm and 50 mm and four-point bending. The experimental program was conducted in the Highbay Laboratory of the School of Civil, Mining and Environmental Engineering at the University of Wollongong, Australia. The details of the experimental program and the results are explained in the following sections.

5.2 Details of specimens

Sixteen circular hollow core short concrete specimens having the dimensions of 206 mm in diameter, 800 mm in height and a 90 mm in diameter hole were made with RPC. These specimens were divided into four groups. The first group was the control group consisting of four unconfined specimens reinforced with six deformed steel bars N12 (12-mm diameter deformed bars) as longitudinal reinforcement. Plain steel bars R10 (10-mm diameter plain bars) were used as helices with a pitch of 50 mm. The design of the steel reinforcement in this study meets the requirements of the Australian Standard
(AS) 3600 (2010) for concrete structures. The specimens of the second group had the same configuration as the first group except they were externally confined with a 1.5 mm thick CFRP tube. The specimens of the third group were composed of outer CFRP tube with a 1.5 mm thickness, inner Polyvinyl chloride (PVC) tube with a 3.5 mm thickness and RPC between the two tubes. Finally, the specimens of the fourth group had no conventional steel reinforcement and they were made with a 1.5 mm thick external CFRP tube, a 3.5 mm thick internal steel tube and RPC in between. In the column design of the fourth group, a steel tube was selected in order to obtain an equivalent axial load capacity to the steel bars that are used in the column design of the other groups.

For the purposes of this study, each specimen is identified with an acronym. The symbol R refers to the use of steel bar reinforcement. The symbol C stands for confinement with a CFRP tube. The symbols P and S refer to the presence of an inner PVC tube and steel tube within the specimen, respectively. Finally, the numbers 0, 25, 50 and the letter B indicate that the specimen is tested under concentric load, 25 mm eccentric load, 50 mm eccentric load and four-point bending, respectively. For example: Specimen CR50 is steel reinforced specimen confined with an external CFRP tube and subjected to 50 mm eccentric load and Specimen CSB is confined with an external CFRP tube, internal steel tube and subjected to four-point bending. The geometry of the specimens is presented in Figure 5.1 and Table 5.1.
Figure 5.1 Cross-section details of the HCRPC specimens
Table 5.1 Experimental test matrix

<table>
<thead>
<tr>
<th>Specimen label</th>
<th>Outer CFRP tube</th>
<th>Inner tube</th>
<th>Internal reinforcement</th>
<th>Test eccentricity (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R0</td>
<td>----</td>
<td>----</td>
<td>6N12 R10 @ 50 mm</td>
<td>0</td>
</tr>
<tr>
<td>R25</td>
<td>206 mm inner Diameter × 1.5 mm wall thickness</td>
<td>----</td>
<td>6N12 R10 @ 50 mm</td>
<td>25</td>
</tr>
<tr>
<td>R50</td>
<td>206 mm inner Diameter × 1.5 mm wall thickness</td>
<td>----</td>
<td>6N12 R10 @ 50 mm</td>
<td>50</td>
</tr>
<tr>
<td>RB</td>
<td>206 mm inner Diameter × 1.5 mm wall thickness</td>
<td>----</td>
<td>6N12 R10 @ 50 mm</td>
<td>Bending</td>
</tr>
<tr>
<td>CR0</td>
<td>206 mm inner Diameter × 1.5 mm wall thickness</td>
<td>----</td>
<td>6N12 R10 @ 50 mm</td>
<td>0</td>
</tr>
<tr>
<td>CR25</td>
<td>206 mm inner Diameter × 1.5 mm wall thickness</td>
<td>----</td>
<td>6N12 R10 @ 50 mm</td>
<td>25</td>
</tr>
<tr>
<td>CR50</td>
<td>206 mm inner Diameter × 1.5 mm wall thickness</td>
<td>----</td>
<td>6N12 R10 @ 50 mm</td>
<td>50</td>
</tr>
<tr>
<td>CRB</td>
<td>206 mm inner Diameter × 1.5 mm wall thickness</td>
<td>----</td>
<td>6N12 R10 @ 50 mm</td>
<td>Bending</td>
</tr>
<tr>
<td>CRP0</td>
<td>206 mm inner Diameter × 1.5 mm wall thickness</td>
<td>PVC of 90 mm outer Diameter × 3.5 mm wall thickness</td>
<td>6N12 R10 @ 50 mm</td>
<td>0</td>
</tr>
<tr>
<td>CRP25</td>
<td>206 mm inner Diameter × 1.5 mm wall thickness</td>
<td>PVC of 90 mm outer Diameter × 3.5 mm wall thickness</td>
<td>6N12 R10 @ 50 mm</td>
<td>25</td>
</tr>
<tr>
<td>CRP50</td>
<td>206 mm inner Diameter × 1.5 mm wall thickness</td>
<td>PVC of 90 mm outer Diameter × 3.5 mm wall thickness</td>
<td>6N12 R10 @ 50 mm</td>
<td>50</td>
</tr>
<tr>
<td>CRPB</td>
<td>206 mm inner Diameter × 1.5 mm wall thickness</td>
<td>PVC of 90 mm outer Diameter × 3.5 mm wall thickness</td>
<td>6N12 R10 @ 50 mm</td>
<td>Bending</td>
</tr>
<tr>
<td>CS0</td>
<td>206 mm inner Diameter × 1.5 mm wall thickness</td>
<td>Steel of 90 mm outer Diameter × 3.5 mm wall thickness</td>
<td>---- ----</td>
<td>0</td>
</tr>
<tr>
<td>CS25</td>
<td>206 mm inner Diameter × 1.5 mm wall thickness</td>
<td>Steel of 90 mm outer Diameter × 3.5 mm wall thickness</td>
<td>---- ----</td>
<td>25</td>
</tr>
<tr>
<td>CS50</td>
<td>206 mm inner Diameter × 1.5 mm wall thickness</td>
<td>Steel of 90 mm outer Diameter × 3.5 mm wall thickness</td>
<td>---- ----</td>
<td>50</td>
</tr>
<tr>
<td>CSB</td>
<td>206 mm inner Diameter × 1.5 mm wall thickness</td>
<td>Steel of 90 mm outer Diameter × 3.5 mm wall thickness</td>
<td>---- ----</td>
<td>Bending</td>
</tr>
</tbody>
</table>

5.3 Materials

The materials used in the experimental work are RPC, longitudinal and helical steel for internal reinforcement, steel tube and PVC tube for internal confinement and CFRP tube for external confinement.

5.3.1 Reactive Powder Concrete (RPC)

In this study, due to the load capacity limitation of the testing machine, the RPC specimens were designed to obtain a compressive strength of 100 MPa at 28 days. Three concrete cylinders with the dimensions of 100 mm in diameter × 200 mm in height were tested at the age of 28 days according to AS 1012.9 (1999) to determine the compressive strength. The RPC was made with commercially available materials:
Bastion Type 1 General Purpose cement that complies with the AS3972 (1997), Sydney sand with range of particle size from 150 µm to 600 µm and Sika-Fume densified silica fume manufactured by Sika Australia. The superplasticizer Viscocrete 3015LF produced by Sika Australia (2015) which complied with ASTM C494 (2015) was used in order to maintain the required workability of the RPC. In addition, straight shape steel fibres supplied by Ganzhou Daye Metallic Fibres (2015) were used in this study, Table 5.2 presents the details of the steel fibres. Table 5.3 shows the mix design of the RPC that was used to cast all specimens. Although 3% of steel fibre showed the highest strength results as presented in Chapter 3 but 2% of steel fibre was used to reinforced the RPC in this study. Because when 3% of steel fibre is used the flowability of the RPC mix dramatically decreased which created some issues associated with mixing and pouring this type of concrete, especially in steel reinforced thin sections (hollow core sections).

Table 5.2 Properties of steel fibre

<table>
<thead>
<tr>
<th>Type</th>
<th>Length of fibre $l_f$ (mm)</th>
<th>Diameter of fibre $d_f$ (mm)</th>
<th>Fibre aspect ratio $\alpha_f (l_f / d_f)$</th>
<th>Ultimate tensile strength $\sigma_{fu}$ (MPa)</th>
<th>Shape</th>
</tr>
</thead>
<tbody>
<tr>
<td>WSF 0213</td>
<td>13</td>
<td>0.2</td>
<td>65</td>
<td>2500</td>
<td>Straight</td>
</tr>
</tbody>
</table>

Table 5.3 Mix design of RPC

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight for 1 m$^3$ (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GP Cement</td>
<td>800</td>
</tr>
<tr>
<td>Silica fume</td>
<td>250</td>
</tr>
<tr>
<td>Fine sand</td>
<td>1050</td>
</tr>
<tr>
<td>Water</td>
<td>180</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>60</td>
</tr>
<tr>
<td>Steel fibres</td>
<td>160</td>
</tr>
</tbody>
</table>

99
An electronic balance was used to weigh all the dry materials that were mixed in a laboratory mixer of 0.1 m$^3$ capacity. Then, the water and superplasticizer were added to the dry mixture. After a period of 10 minutes of mixing, the full amount of steel fibre was added and the desired flowability (Flow table test $>$120 mm) was obtained in accordance with ASTM C230 (2014), as shown in Figure 5.2. The fresh RPC was then placed into the formwork that consisted of four PVC pipes and twelve CFRP tubes vertically fixed on a wooden base. During the process of concrete placing, an electric vibrator was used to compact and eliminate air voids. Wet hessian and plastic sheets were used for curing and covering the specimens for a period of 28 days. At the age of 28 days, the average compressive strength was 105 MPa.

![Figure 5.2 Flow table test of the RPC flowability](image)

### 5.3.2 Steel reinforcement

The internal reinforcement that consisted of six deformed N12 bars were placed in the longitudinal direction with a diameter of 12 mm, total length of 760 mm and nominal tensile strength of 500 MPa. For the transverse direction, helical plain steel R10 bars were used with a diameter of 10 mm and nominal tensile strength of 250 MPa. The mechanical tensile properties of steel were determined according to AS 1391 (2007).
Three samples were taken from each diameter and tested using the Instron universal testing machine, as can be seen in Figure 5.3. Each sample had a total length of 500 mm, 340 mm clear testing length and 80 mm gripping length at each end of the sample. The typical stress-strain behaviours of the steel bars (N12 and R10) are shown in Figure 6.4. For N12 steel bars, the modulus of elasticity, the yield tensile strength and the corresponding strain were 190 GPa, 560 MPa and 0.003 mm/mm, respectively. For R10 steel bars, the modulus of elasticity, the yield tensile strength and the corresponding strain were 180 GPa, 340 MPa and 0.0019 mm/mm, respectively.

Figure 5.3 Test setup of the steel bars tensile test
5.3.3 CFRP tube

The filament-wound CFRP tubes of 1.5 mm thickness were manufactured by Composite Spars and Tubes Company based in Caringbah, NSW, Australia (CST 2015). These tubes consisted of two layers. The inner layer (0.5 mm thickness) was orientated in the longitudinal direction with a 0º winding angle, while the outer layer (1.0 mm thickness) was orientated in the hoop direction with an 89º winding angle.

Three samples of CFRP coupons with a 250 mm total length, a 138 mm test length and a 25 mm width, were cut out of the longitudinal direction of the tube to determine the CFRP tensile properties. A gripping length of 56 mm at both ends of the CFRP coupon was used to apply the tensile load on the samples. The CFRP coupon test was conducted according to the standard ASTM D3039 (2014). Figure 5.5 shows the dimensions of the CFRP coupon. The Instron universal testing machine with a loading capacity of 500 kN was used to obtain the tensile properties of the CFRP coupons, as can be seen in Figure 5.6. Based on the test results of the three samples, the average values of ultimate tensile
strength, modulus of elasticity and ultimate tensile strain were 604 MPa, 46 GPa, and 1.35%, respectively. Figure 5.7 shows the tensile stress-strain behaviour of the CFRP tube in the longitudinal direction.

Figure 5.5 Dimensions of CFRP coupon

Figure 5.6 Tensile test setup for CFRP coupons
In order to determine the tensile properties of the CFRP tube in the hoop direction, tensile split-disk tests were conducted on three CFRP rings, as shown in Figure 5.8. The CFRP rings with dimensions of 35 mm in width and 1.5 mm in thickness were cut from the same CFRP tube. The split-disk test was conducted in accordance with the ASTM D2290 (2012) standard. Six strain gauges with 5 mm of gauge length were attached to the outer face of the CFRP disk. Two strain gauges were located at the two gaps, while the other four gauges were attached at 25 mm away from the gaps. Due to the bending effect at the two gaps, the results of the two strain gauges there were lower than the results of the other strain gauges. As expected, all the CFRP rings showed a brittle failure with a rupture at the disk gap, Figure 5.9 shows the typical failure of CFRP disk. The average values of ultimate tensile strength, modulus of elasticity and ultimate tensile strain were 1160 MPa, 86 GPa, and 1.31%, respectively. Figure 5.10 shows the stress-strain behaviour of the CFRP tube in the hoop direction.
Figure 5.8 Test setup for tensile split-disk tests of CFRP tube

Figure 5.9 Typical failure for CFRP disk
5.3.4 PVC tube

The PVC tube with an inner diameter of 90 mm and a thickness of 3.5 mm was used in this study. Three coupons having the dimensions of 165 mm in total length, 57 mm in test length and 13 mm in test width were taken from the longitudinal direction of the PVC tube to obtain the tensile stress-strain relationship of this material according to ASTM D638 (2014). Figure 5.11 shows the dimensions of the PVC coupon. The Instron testing machine with load capacity of 150 kN was used to obtain the tensile stress of the PVC coupons, whereas the tensile strain was obtained from a clip-on extensometer attached to the specimen, as can be seen in Figure 5.12. The average values of the ultimate tensile strength, ultimate strain and the modulus of elasticity were 53.4 MPa, 43.7% and 4.1 GPa, respectively. Figure 5.13 shows the tensile stress-strain behaviour of the PVC coupons.

Figure 5.10 Stress-strain behaviour of the CFRP tube in the hoop direction
Three samples of the PVC tube with a length of 800 mm were tested in axial compression to determine the maximum axial load capacity of the PVC tube, as can be seen in Figure 5.14. The average maximum axial load of the PVC tubes under compression was 54 kN.

Figure 5.11 Dimensions of the PVC coupon

Figure 5.12 Tensile test setup for PVC coupons
Figure 5.13 Tensile stress-strain behaviour of the PVC coupons

Figure 5.14 PVC tube under pure axial compression
5.3.5 Steel tube

In this study, steel tubes with an inner diameter of 90 mm and a thickness of 3.5 mm were used. Tensile tests on three steel coupons were extracted from one batch of steel tube. As shown in Figure 5.15, the coupons having the dimensions of 300 mm in total length, 120 mm in test length and 20 mm in test width were cut from the steel tube along the longitudinal direction and were tested according to the AS 1391.07 (2007). Figure 5.16 shows the typical tensile stress-strain behaviour of the steel tube. The average values of the modulus of elasticity, yield strength, and ultimate tensile strength of the steel tubes were 200 GPa, 430 MPa and 500 MPa, respectively.

The peak axial load of the steel tube was determined by testing three samples (800 mm in length) of the steel tube under pure axial compression, as can be seen in Figure 5.17. The average peak axial load of the steel tubes was 320 kN. The steel tube failed with global buckling at the mid-height of the tube accompanied with local deformation at both ends of the tube, as can be seen in Figure 5.17b.

![Figure 5.15 Dimensions of the steel coupon](image-url)
Figure 5.16 Typical tensile stress-strain behaviour of steel tube

Figure 5.17 Steel tube under pure axial compression: (a) Test setup and (b) Steel tube after failure
5.4 Fabrication of specimens

Four PVC pipes with an internal diameter of 206 mm and total length of 800 mm were used as formworks to cast the specimens of Group R. In addition, twelve CFRP tubes with an internal diameter of 206 mm and total length of 800 mm were used as stay-in-place formworks to cast the rest groups of specimens. All formworks were placed and fixed vertically by using a frame of timber as shown in Figure 5.18. The steel reinforcement cages were made of six N12 bars with a length of 760 mm, in order to have 20 mm concrete cover from the top and the bottom of the specimen. The steel reinforcement cages were also included transverse steel helix having a pitch of 50 mm and an outer diameter of 170 mm. A concrete cover of 18 mm was made to the face of the transverse reinforcement. Figure 5.19 shows the assembly of the steel reinforcement cages.

Eight foam cylinders with an outer diameter of 90 mm and a total length of 800 mm were used to create the inner hole in the specimens of Group R and Group CR. For specimens in Group CRP and Group CS, PVC tubes and steel tubes with an outer diameter of 90 mm and total length of 800 mm were used, respectively, as a stay-in-place formwork to create an inner hole and provide internal confinement to the specimens.

The RPC was poured into each formwork in four levels and at each level it was vibrated with cordless needle vibrator to compact the concrete and eliminate the air bubbles. Then, all specimens were covered with plastic sheet and wet hessian to ensure moist curing condition for all specimens. This curing condition was maintained for 28 days prior to the test of the specimens.
Figure 5.18 PVC and CFRP tubes formworks

Figure 5.19 Steel reinforcement cages
5.5 Testing of specimens

In order to obtain the load-deformation test results of the hollow RPC specimens, a Denison testing machine with a loading capacity of 5000 kN was used. To prevent premature failure of the column ends during the test, a single layer of CFRP sheet with a width of 100 mm was used to wrap the top and the bottom of the column specimens. In all loading cases of the specimens, the results of axial deformation were recorded with two Linear Variable Differential Transformers (LVDTs) attached to the lower loading head of the testing machine, as shown in Figure 5.20. The axial load was recorded by a load cell placed at the bottom of the testing machine. The specimens were preloaded up to 5% of the estimated load carrying capacity to prevent minor movements between the loading heads of the testing machine and the specimen, and then the load was released to 20 kN before starting the test. During the test, the load was applied with a displacement rate of 0.3 mm/minute until the resistance of the specimens dropped to 30% of the peak load. The LVDTs and the load cell were connected to a data logger to record the readings every two seconds.

For eccentrically loaded specimens, the loading heads were adjusted to provide an eccentricity of 25 mm and 50 mm, as shown in Figure 5.21. The lateral displacement was measured by using a laser triangulation that was located at the mid-height of the specimen. The axial load and axial deformation were recorded using the same instrumentation of concentrically loaded specimens.

Four specimens were tested under four-point bending. Two rigs were placed on the top and bottom of the specimens to transfer the applied load from the testing machine to the specimen. The clear span between the supports was 700 mm and the distance between the upper point loads was 230 mm. The typical test setup of the beam specimens is shown in Figure 5.22. The midspan deflection of the specimens was measured using
laser triangulation. The loading rate and data recording were the same as column specimen testing.

Figure 5.20 Typical setup of concentric loading test

Figure 5.21 Typical setup of eccentric loading test

Detail A
5.6 Experimental results and discussion

5.6.1 Mode of failure

All specimens were subjected to monotonic load until failure. Failure of unconfined HCRPC columns was evident in the gradual cracking near the mid-height of the column specimens. Spalling of the concrete cover was followed by the buckling of the longitudinal steel bars outwards. The failure of unconfined HCRPC specimens after the ultimate load was sudden but not explosive, because of using steel fibre within the concrete mix. For the CFRP-confined HCRPC specimens, the failure was noticed physically by the occurrence of CFRP ripples on the surface of the CFRP tube in the hoop direction followed by snapping sounds which were heard subsequently prior to the ultimate failure due to the strap-by-strap laceration of CFRP fibre within the external CFRP tube. In general, unconfined HCRPC specimens showed a brittle failure in contrast with those with CFRP tube confinement that showed a ductile failure mechanism. Figures 5.23 and 5.24 show the typical failure modes of confined and unconfined HCFRP specimens, respectively.
Figure 5.23 Typical failure modes of unconfined specimens

Figure 5.24 Typical failure modes of confined specimens
5.6.2 Hollow core RPC specimens under concentric load

Four HCRPC column specimens of different configurations were tested under uniform concentric load until failure. Figure 5.25 illustrates the axial load-axial deformation behaviour of the four concentrically tested specimens.

Specimen R0 showed lower load and axial deformation than the FRP confined column specimens. The failure of this specimen was recognised by continuous concrete crack propagation at the mid-height of the concrete but the concrete cover did not spall off due to the presence of steel fibre. After carrying a load of 2986.9 kN, Specimen R0 experienced a sudden drop in the axial load, which indicates the brittle failure of this specimen. An ultimate axial deformation of 5.0 mm was recorded.

Specimen CR0 carried a maximum load of 3360.2 kN which is higher than the load carried by Specimen R0 due to CFRP tube confinement. In addition, the ultimate axial deformation of Specimen CR0 dramatically increased to 16.5 mm. The axial load-axial deformation behaviour of Specimen CR0 consists of two parts. The first part is the linear behaviour up to the maximum load. Then, in the second part, the CFRP tube experienced multi CFRP strap ruptures in different locations within the mid-height of the specimen, causing axial load fluctuation. This behaviour ended with a sudden drop of axial load after the ultimate load was reached.

As shown in Figure 5.25, Specimen CRP0 sustained the highest values of axial load and deformation among the other concentrically loaded specimens. Specimen CRP0 was externally confined with the CFRP tube and internally confined with the PVC tube, the second part of the load-deformation curve showed an ascending branch up to an ultimate axial load of 3718.8 kN. For the same reason, the reading of the ultimate axial deformation continued to increase, recording 18.7 mm.
The axial load-axial deformation curve of Specimen CS0 showed a different behaviour in the second branch of the curve compared to other specimens. Multiple peaks of axial load can be seen along the second branch. These peaks of the axial load refer to the rupture of the CFRP straps one by one in the hoop direction. For Specimen CS0, the longitudinal steel bars and the helix were replaced with a steel tube of an equivalent axial load capacity located inside the hollow core. Thus, an axial load of 3346.1 kN was obtained by Specimen CS0, which was nearly the same axial load of Specimen CR0. However, by using a steel cage of longitudinal bars and helix within the section of Specimen CR0, the axial load in the second branch showed less fluctuation than the axial load of Specimen CS0.

Figure 5.25 Axial load-axial deformation diagrams of concentrically tested specimens

In this study, the ductility of the specimens was determined by using a method suggested by Park (1989). Figure 4.20 in Chapter 4 explains how the yield and ultimate points are determined. The axial load, axial deformation and ductility of the specimens under concentric load are shown in Table 5.4.
8 specimens were tested under eccentric loading, the first four specimens with an eccentricity of 25 mm and the second four specimens with an eccentricity of 50 mm. Figure 5.27 presents the axial and lateral deformation versus the axial load of the specimens subjected to a load eccentricity of 25 mm. All the specimens that were tested under 25 mm eccentric load failed in compression. It can be seen from Figure 5.19 that the highest maximum load of 2290.5 kN was sustained by Specimen CRP25. Figure 5.27, also shows that the maximum axial load of Specimens CR25, CRP25 and CS25 was enhanced by 7.6%, 13.3% and 5.5%, respectively compared to the maximum load of Specimen R25 (unconfined specimen). Compared to Specimen R25, the ultimate axial deformation was dramatically increased by 279%, 357% and 272% for Specimens CR25, CRP25 and CS25, respectively. Figure 5.27 also shows that the lateral deformations of 25 mm eccentric loaded specimens are higher than the axial deformations. Table 5.5 presents the test results of the load, axial and lateral deformations and ductility of specimens under 25 mm eccentric load.

5.6.3 Hollow core RPC specimens under eccentric load

<table>
<thead>
<tr>
<th>Specimens</th>
<th>R0</th>
<th>CR0</th>
<th>CRP0</th>
<th>CS0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum load (kN)</td>
<td>2986.9</td>
<td>3360.2</td>
<td>3717.4</td>
<td>3346.1</td>
</tr>
<tr>
<td>Axial deformation at maximum load (mm)</td>
<td>5.0</td>
<td>11.8</td>
<td>17.7</td>
<td>11.0</td>
</tr>
<tr>
<td>Yield load (kN)</td>
<td>2645.6</td>
<td>2849.0</td>
<td>2976.1</td>
<td>2744.7</td>
</tr>
<tr>
<td>Axial deformation at yield load (mm)</td>
<td>3.7</td>
<td>4.3</td>
<td>4.7</td>
<td>4.0</td>
</tr>
<tr>
<td>Ultimate axial deformation (mm)</td>
<td>5.0</td>
<td>16.5</td>
<td>18.7</td>
<td>15.9</td>
</tr>
<tr>
<td>Ductility</td>
<td>1.29</td>
<td>3.84</td>
<td>5.45</td>
<td>3.63</td>
</tr>
</tbody>
</table>
Figure 5.27 Axial load-deformation diagrams for column specimens tested under 25 mm eccentricity.

Table 5.5 Experimental results of specimens tested under 25 mm eccentric loads

<table>
<thead>
<tr>
<th>Specimen</th>
<th>R25</th>
<th>CR25</th>
<th>CRP25</th>
<th>CS25</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum load (kN)</td>
<td>2021.5</td>
<td>2176</td>
<td>2290.5</td>
<td>2132.6</td>
</tr>
<tr>
<td>Axial deformation at maximum load (mm)</td>
<td>4.7</td>
<td>5.4</td>
<td>7.5</td>
<td>6.1</td>
</tr>
<tr>
<td>Lateral deformation at max. load (mm)</td>
<td>2.7</td>
<td>6.27</td>
<td>6.7</td>
<td>3.69</td>
</tr>
<tr>
<td>Yield load (kN)</td>
<td>1768.3</td>
<td>2051.7</td>
<td>2119.4</td>
<td>2018.6</td>
</tr>
<tr>
<td>Axial deformation at yield load (mm)</td>
<td>3.9</td>
<td>4.6</td>
<td>5.0</td>
<td>4.2</td>
</tr>
<tr>
<td>Lateral deformation at yield load (mm)</td>
<td>2.0</td>
<td>2.8</td>
<td>3.7</td>
<td>2.9</td>
</tr>
<tr>
<td>Ultimate axial deformation (mm)</td>
<td>4.9</td>
<td>18.3</td>
<td>22.4</td>
<td>18.6</td>
</tr>
<tr>
<td>Ultimate lateral deformation (mm)</td>
<td>2.7</td>
<td>33.7</td>
<td>38.9</td>
<td>25.4</td>
</tr>
<tr>
<td>Ductility</td>
<td>1.34</td>
<td>4.01</td>
<td>5.48</td>
<td>4.31</td>
</tr>
</tbody>
</table>
Figure 5.28 illustrates the axial and lateral deformation versus the maximum axial load of the specimens subjected to load eccentricity of 50 mm. The highest maximum load of 1572.1 kN was achieved with Specimen CRP50. Based on the test results presented in Figure 6.28, the maximum axial load of Specimens CR50, CRP50 and CS50 was slightly increased by 4.9%, 10.8% and 2.4%, respectively compared to Specimen R50 (unconfined specimen). The ultimate axial deformation was significantly increased by 357%, 428% and 471% for Specimens CR50, CRP50 and CS50, respectively compared to Specimen R50 (unconfined specimen). The test results of the load, axial and lateral deformations and ductility of specimens tested under 50 mm eccentric load are presented in Table 5.6.

Figure 5.28 Axial load-deformation diagrams for column specimens tested under 50 mm eccentricity
Table 5.6 Experimental results of specimens tested under 50 mm eccentric loads

<table>
<thead>
<tr>
<th>Specimen</th>
<th>R50</th>
<th>CR50</th>
<th>CRP50</th>
<th>CS50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum load (kN)</td>
<td>1418.9</td>
<td>1488.2</td>
<td>1572.1</td>
<td>1452.3</td>
</tr>
<tr>
<td>Axial deformation at maximum load (mm)</td>
<td>4.2</td>
<td>6.7</td>
<td>4.5</td>
<td>5.2</td>
</tr>
<tr>
<td>Lateral deformation at max. load (mm)</td>
<td>4.1</td>
<td>2.5</td>
<td>6.4</td>
<td>2.8</td>
</tr>
<tr>
<td>Yield load (kN)</td>
<td>1236.5</td>
<td>1378.9</td>
<td>1521.1</td>
<td>1401.7</td>
</tr>
<tr>
<td>Axial deformation at yield load (mm)</td>
<td>3.9</td>
<td>4.1</td>
<td>4.3</td>
<td>4.7</td>
</tr>
<tr>
<td>Lateral deformation at yield load (mm)</td>
<td>3.2</td>
<td>2.1</td>
<td>5.3</td>
<td>2.3</td>
</tr>
<tr>
<td>Ultimate axial deformation (mm)</td>
<td>4.2</td>
<td>20.5</td>
<td>25.2</td>
<td>23.5</td>
</tr>
<tr>
<td>Ultimate lateral deformation (mm)</td>
<td>4.1</td>
<td>35.5</td>
<td>32.1</td>
<td>27.8</td>
</tr>
<tr>
<td>Ductility</td>
<td>1.10</td>
<td>4.71</td>
<td>5.82</td>
<td>5.15</td>
</tr>
</tbody>
</table>

Compared to the specimens in Group R, the maximum axial load of specimens in Groups CR, CRP, and CS was observed to decrease with the increase of loading eccentricity. On the other hand, the ultimate axial deformation of specimens in Groups CR, CRP, and CS was observed to increase dramatically by increasing the load eccentricity. Thus, higher values of ductility were achieved by 50 mm eccentric loaded specimens compared to those specimens tested under 25 mm eccentricity.

### 5.6.4 Hollow core RPC specimens under pure bending loading

In order to determine the maximum bending moment of the specimens, a flexural test was performed under a four-point bending system. Figure 5.29 shows the load versus midspan deflection curves of the four specimens. According to this figure, the highest values of load, corresponding mid-span deflection and ductility were achieved by Specimen CRPB. In comparison with Specimen RB (unconfined specimen), the maximum load of Specimens CRB, CRPB and CSB were increased by 133.1%, 138.5% and 78.4%, respectively. These increments were due to the effect of the longitudinal FRP fibres within the CFRP tube that significantly enhanced the load carrying capacity.
and ductility of the specimens. It should be mentioned that during test of Specimens CRPB and CSB, there was a very small relative movement between the internal tubes (PVC and steel) and the surrounding RPC. Table 5.7 presents the test results of the four beam specimens, including the results of ductility. These results of the ductility were calculated using the same methods as these used above for concentrically and eccentrically loaded specimens.

Table 5.7 Experimental results of specimens tested under flexural loads

<table>
<thead>
<tr>
<th>Beam specimen</th>
<th>RB</th>
<th>CRB</th>
<th>CRPB</th>
<th>CSB</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum load (kN)</td>
<td>340.0</td>
<td>792.7</td>
<td>811.0</td>
<td>606.4</td>
</tr>
<tr>
<td>Midspan deflection at maximum load (mm)</td>
<td>7.2</td>
<td>27.2</td>
<td>29.9</td>
<td>23.8</td>
</tr>
<tr>
<td>Yield load (kN)</td>
<td>284.6</td>
<td>579.7</td>
<td>651.6</td>
<td>443.4</td>
</tr>
<tr>
<td>Midspan deflection at yield load (mm)</td>
<td>4.3</td>
<td>6.4</td>
<td>5.5</td>
<td>4.9</td>
</tr>
<tr>
<td>Ultimate midspan deflection (mm)</td>
<td>15.8</td>
<td>30.0</td>
<td>32.3</td>
<td>24.8</td>
</tr>
<tr>
<td>Ductility</td>
<td>3.33</td>
<td>6.22</td>
<td>6.60</td>
<td>4.21</td>
</tr>
</tbody>
</table>

Figure 5.29 Load-midspan deflection diagrams for beam specimens tested under four-point bending
5.6.5 Effect of CFRP tube confinement

The effect of the external confinement of the CFRP tube on the strength and ductility of HCRPC specimens was experimentally investigated by comparing the test results obtained from the specimens of Groups R and CR. Figure 5.30 shows the normalized values of axial maximum load and ductility of specimens in Group CR with respect to the ones in Group R. For concentrically loaded specimen, the maximum axial load and ductility of Specimen CR0 were increased by 12.5% and 198%, respectively compared to Specimen R0 (unconfined column).

For eccentrically loaded specimens, the maximum load of Specimens CR25 and CR50 was increased by 7.6% and 7.5%, respectively compared to the corresponding unconfined specimens. In addition, the ductility of Specimens CR25 and CR50 was also increased by 200% and 328%, respectively. For flexural loading, the CFRP layer in the longitudinal direction has a significant influence on the maximum load and ductility of the specimen. The maximum load and the ductility of Specimen CRB increased by 133.1% and 86.7%, respectively compared to the corresponding unconfined specimens.

Based on the test results presented in Figure 6.30, it can be seen that the use of CFRP tube can significantly increase the ductility of HCRPC specimens but the maximum load of the confined specimen increased slightly due to the existence of the inner hole.
5.6.6 Effect of internal confinement with PVC tube

The effect of using PVC tubes for inner confinement on the strength and ductility of HCRPC specimens can be ascertained by comparing the test results of specimens in Group CR and Group CRP. Figure 5.31 shows the normalized values of maximum axial load and ductility of the specimens in Group CRP with respect to those in Group CR. In terms of maximum axial load and under concentric loading, Specimen CRP0 showed an increase of 10.1% compared to Specimen CR0. It can be seen from Figure 5.25 that the second branch of the load-deformation curve of Specimen CRP0 experienced an ascending behaviour because of the internal confinement provided by the PVC tube. The maximum load was also higher for Group CRP than Group CR under 25 mm, 50 mm eccentric loading. Figure 5.31 shows the normalized maximum axial load and normalized ductility of the specimens in Group CR and Group CRP under different loading conditions.
According to this figure, the ductility of Specimens CRP0, CRP25, CRP50 and CRPB was increased by 42%, 37%, 24% and 6% compared to Specimens CR0, CR25, CR50 and CRB, respectively. These findings indicate that introducing PVC tube in HCRPC specimens for internal confinement can slightly enhanced the strength but the ductility was dramatically improved for this type of structural members.

![Graph showing normalized maximum load and ductility](image)

**Figure 5.31** Effect of inner PVC tube on the maximum load and ductility of CFRP-confined HCRPC specimens

**5.6.7 Effect of replacing normal steel reinforcement with steel tube**

The effect of using steel tube as an alternative to the conventional steel reinforcement was investigated by comparing the experimental results of the specimens in Group CR and Group CS. Figure 5.32 shows the normalized values of maximum axial load and ductility of the specimens in Group CS with respect to those in Group CR. In the column design of Group CS, the steel tube was selected to obtain an equivalent axial load to the steel bars that were used in the columns design of Group R.
The test results showed that the maximum axial load was nearly the same under concentric and eccentric loading conditions but under flexural loading, Specimen CRB showed higher maximum axial load and ductility than Specimen CSB.

Figure 5.25 shows the load-deformation curves of Specimens CR0 and CS0. For concentrically loaded specimens, Specimen CR0 showed less fluctuation of load-deformation behaviour post the yield load than Specimen CS. The reason for this is the presence of the steel bar and helix within the concrete section of Specimen CR0 that provide an additional confinement to the concrete that minimizes and distributes the applied lateral pressure on the outer CFRP tube. On the other hand, the presence of the inner steel tube within the section of Specimen CS0 provides an internal confinement that increases the outward concrete pressure on the surrounding CFRP tube.

![Figure 5.32 Effect of replacing normal steel reinforcement with steel tube on the maximum load and ductility of CFRP-confined HCRPC specimens](image-url)
5.7 Summary

This chapter has presented the experimental test results of sixteen specimens that explain the behaviour of HCRPC specimens with and without CFRP tube confinement in different configurations. These specimens were tested under concentric load, 25 mm eccentric load, 50 mm eccentric load and four-point bending. The test results involved the interpretation of the failure mode, axial load versus axial and lateral deformation behaviour and the ductility of the specimens. Based on the experimental test results presented above, the following conclusions can be drawn:

- By introducing CFRP tube confinement, the strength of HCRPC specimens was slightly increased, whereas the ductility was significantly improved.

- By increasing the eccentricity of loading, the maximum axial load of all CFRP-confined HCRPC specimens having different configurations decreased in comparison with the unconfined HCRPC specimen. On the other hand, the axial deformation capacity of all CFRP-confined specimens was observed to increase dramatically with the increase of loading eccentricity. Thus, higher values of ductility were achieved by the 50 mm eccentric loaded specimens compared to those specimens tested under 25 mm eccentricity.

- The four-point bending test indicates that the use of a CFRP tube can significantly increase the maximum axial load and ductility of HCRPC specimens.

- By replacing the conventional steel reinforcement with an equivalent steel tube within the section of HCRPC specimens, the values of strength and ductility are nearly the same. However, under flexural loading a better performance of HCRPC specimen with normal steel reinforcement can be achieved than the one with the steel tube.
By providing an inner PVC tube to the HCRPC specimens, which are internally reinforced with conventional steel reinforcement and externally confined with a CFRP tube, the strength was slightly enhanced but the ductility was dramatically improved for this type of structural members. This is because of the beneficial effect of the PVC tube that provides an additional inner confinement to the annular concrete section.
6 ANALYTICAL STUDY AND DISCUSSION OF RESULTS

6.1 Introduction

This chapter presents an analytical program to create axial load-bending moment \((P-M)\) interaction diagrams for the CFRP-confined Hollow Core Reactive Powder Concrete (HCRPC) specimens that had been presented in Chapter 5. A numerical integration approach was used to calculate the theoretical values of the \(P-M\) diagrams and a computer program of MS Excel was also used for this purpose. The analytical program in this chapter shows in details the modelling of the behaviour of each component in the HCRPC specimens such as the CFRP-confined RPC, longitudinal steel bars, longitudinal CFRP tube, PVC tube and steel tube. This chapter also includes a parametric study that was carried out to examine the effect of using an external CFRP tube confinement, using an internal PVC or steel tube on the analytical \(P-M\) interaction diagrams of the HCRPC specimens.

6.2 Assumptions

In order to simplify the analysis of the HCRPC sections, common assumptions are adopted to derive the analytical axial load-bending moment interaction diagrams. These assumptions are presented below:

1. Plane sections remain plane after deformation. This means strain varies linearly across the section of the member. This assumption simplifies the calculations with minor errors but for structural design purposes, it can be neglected.

2. Strain compatibility among the RPC, steel bars, CFRP tube, steel tube and PVC tube is assumed, which means a perfect bond among the combined materials of
the HCRPC specimens. This assumption was adopted as the relative movement between the internal tubes (PVC and steel) and the surrounded concrete was very small as mentioned in Chapter 5 above.

3. Tensile strength of the RPC and confinement effect of the steel helix are neglected in the analytical calculations. As mentioned above in Chapter 3, the contribution of the RPC with 2% of steel fibre was very small in changing the tensile behaviour of the RPC. In FRP-confined steel reinforced concrete columns, the steel helix under axial loading remains ineffective until the FRP rupture and for this reason the effect of the steel helix was ignored in the analytical study.

4. The RPC is assumed to fail at the unconfined compressive strength \( f'_{cc} \) and corresponding axial strain \( \varepsilon_{co} \). For the RPC confined with CFRP tube, FRP-confined concrete models presented in Chapter 2 are used to determine the values of the confined compressive strength \( f'_{cc} \) and the corresponding axial strain \( \varepsilon_{cc} \), respectively.

5. Elastic-perfectly plastic behaviours are assumed for the reinforcing steel bars, PVC tube and steel tube under both tension and compression stresses. This behaviour was confirmed with the materials' tests presented in Chapter 5 above.

6. The stiffness of CFRP tubes in the axial and hoop directions are neglected under compression. The compressive strength of the FRP materials have also minor contribution to the axial load capacity and design codes (e.g. ACI 440.1R-06) recommended neglecting this contribution. It is assumed that the CFRP tubes show a linear stress-strain relationship under tensile stress up to the failure. This assumption is based on the material test presented in Chapter 5 above.
7. The ultimate compressive strain of the unconfined concrete is 0.004. This assumption was based on the experimental results of the compressive stress-strain curve of the RPC that presented in Chapter 3 above.

8. The stress, strain, and force are positive in compression and negative in tension.

6.3 Columns geometry and material response

Figure 6.1 illustrates a typical cross-section of the HCRPC that was used to generate the $P-M$ diagrams. The RPC which is the main part of this section is assumed to have a compressive strength of $f'_c$ and corresponding axial strain of $\epsilon_c$ that have been determined by testing solid cylinder specimens (150 mm × 300 mm), see Figure 3.9 in Chapter 3. The HCRPC specimens have a total height of $H$, an inner diameter $D_i$, an outer diameter $D_o$ and a clear concrete cover $cc$ which is the distance from the transverse steel helix to the surface of the specimen.

The longitudinal steel bars reinforcement is assumed to have a number of $n_s$ with a nominal diameter of $d_s$ and a yield tensile strength of $f_{ys}$. For the HCRPC specimens with inner steel tube (Group CS), the values of the longitudinal and transverse reinforcement properties are taken zero.

The HCRPC specimen is assumed to be confined with CFRP tube that consisted of CFRP layers in the hoop and the longitudinal directions. The CFRP layer has a thickness of $t_{f,h}$ in the hoop direction and a thickness of $t_{f,l}$ in the longitudinal direction. The CFRP layer has a tensile strength of $f_f$ and corresponding axial strain of $\epsilon_f$ which are determined by CFRP-coupon tensile test. For unconfined HCRPC specimens the values of CFRP properties are taken zero.
The internal tube (PVC or steel) has a thickness of \( t \). Also, the internal tube is assumed to have a yield tensile strength of \( f_{y,t} \), which is determined by coupon tensile tests.

6.3.1 Calculation of RPC response

In general, the response of concrete is determined by using the approach of an equivalent rectangular stress block. However, this approach was not used in this study due to the unknown application point of the resultant force on a hollow core circular cross-section. Instead, a layer-by-layer numerical integration approach was used to facilitate the calculations where the hollow core cross-section was divided into small thickness parallel layers as shown in Figure 6.2. Each layer has a thickness of \( \Delta h \) and a width of \( b \) and the total number of the layers is \( n_{\text{layers}} \) which can be calculated by dividing the outer diameter of the cross-section by the thickness of the layer. For each layer, the position from the top and the position from the centreline are calculated by using Equation 6.1 and Equation 6.2, respectively.
For each layer, the axial strain of the concrete is assumed to be uniform throughout the layer. Subsequently, the value of the concrete axial strain at the centre of each layer \( \varepsilon_{clayer}(L) \) can be calculated by using Equation 6.3 below:

\[
\varepsilon_{clayer}(L) = \varepsilon_{cu} - \frac{\varepsilon_{cu}}{d_N} dc_{top}(L)
\]

where \( d_N \) is the depth of the neutral axis and \( \varepsilon_{cu} \) is the ultimate axial strain of the concrete. For unconfined RPC specimens (Group R), the value of \( \varepsilon_{cu} \) is assumed to be equal to 0.004. Then, the axial stress \( (\sigma_c) \) at the centre of each layer corresponding to the axial strain \( (\varepsilon_c) \) was determined by a concrete stress-strain model. For Group R, a stress-strain model proposed by Yang et al. (2014) was used in study.
Whereas, a stress-strain model proposed by Yazici and Hadi (2012) was adopted to calculate the axial stress of each layer for the CFRP-confined HCRPC specimens (Group CR), see Equation 2.40 in Chapter 2. In addition, for the CFRP-confined HCRPC specimens with an internal tube (Group CRP and Group CS), a stress-strain model proposed by Xiao et al. (2010) was adopted to calculate the response of the RPC, see Equation 2.40 in Chapter 2. Accordingly, the force of the concrete in the centre of each layer \( F_c(L) \) was calculated by Equation 6.4.

\[
F_c(L) = f_c \times A(L)
\]  

(6.4)

where \( A(L) \) is the area of concrete layer which can be calculated by multiplying the thickness of the layer \( \Delta h \) by its width \( b(L) \). For hollow core circular cross-section, the width of each layer is different from layer to another due to the circular shape and existence of the hollow part within the cross-section, as shown in Figure 6.3. Thus, Equations 6.5 and 6.6 were used to calculate the width of each layer depending on its position.

![Figure 6.3 A concrete layer on a hollow core circular cross-section](image-url)
for $0 \leq d_{centre}(L) \leq D_i$

$$b \ (L) = 2 \left[ \sqrt{\left( \frac{D_o}{2} \right)^2 - \left( d_{centre}(L) \right)^2} - \sqrt{\left( \frac{D_i}{2} \right)^2 - \left( d_{centre}(L) \right)^2} \right]$$  \hfill (6.5)

for $D_i \leq d_{centre}(L) \leq D_o$

$$b \ (L) = 2 \sqrt{\left( \frac{D_o}{2} \right)^2 - \left( d_{centre}(L) \right)^2}$$  \hfill (6.6)

where $L = 1, 2, 3, \ldots, n_{\text{layers}}$

The total force response of the concrete cross-section is calculated by taking the summation of the force for all the concrete layers under compression.

The moment response in the centre of each layer about the centreline of the cross-section was calculated using Equation 6.7.

$$M_c(L) = F_c(L) \times d_{centre}(L) \quad L = 1, 2, 3, \ldots, n_{\text{layers}}$$  \hfill (6.7)

The total moment response of the concrete cross-section is calculated by taking the summation of the moment for the concrete layers above the neutral axis and the tensile strength of the RPC was ignored.
6.3.2 Calculation of longitudinal steel reinforcement response

The six longitudinal steel bars were placed in four layers within the circular cross-section of the specimens as shown in Figure 6.4.

For the first and the fourth layers of steel bars, the position of the steel bar to the centreline \( (ds_{centre1}) \) is equal to \( (rs) \) which is the radius from the centre of the cross-section to the centre of the steel bar and can be determined by Equation 6.8.

\[
ds_{centre1} = r_s = \frac{D_o}{2} - cc - d_n - \frac{d_s}{2}
\]  

(6.8)
For the second and the third layers of steel bars, the position of the steel bar to the centreline \( (d_{\text{centre2}}) \) was calculated by Equation 6.9 as below:

\[
\sin \theta = \frac{d_{\text{centre2}}}{r_s} \quad \Rightarrow \quad d_{\text{centre2}} = \sin 30^\circ \times r_s \quad \Rightarrow \quad d_{\text{centre2}} = \frac{r_s}{2} \quad (6.9)
\]

Thus, the distance from each layer of longitudinal steel bars to the extreme concrete compression fibre was calculated as follow:

\[
\text{for the first and the fourth layers} \quad d_{\text{top1}} = d_{\text{top4}} = \frac{D_o}{2} - r_s \quad (6.10)
\]

\[
\text{for the second and the third layers} \quad d_{\text{top2}} = d_{\text{top3}} = \frac{D_o}{2} - d_{\text{centre2}} \quad (6.11)
\]

The calculation of the axial strain on each steel bar is in a similar way of calculation the axial strain of the concrete layers. For a given applied load and a depth of the neutral axis, the axial strain on each layer of steel bars \( (\varepsilon_{s,\text{layer}}) \) was calculated using Equation 6.12.

\[
\varepsilon_{s,\text{layer}} (L) = \varepsilon_{cu} - \frac{\varepsilon_{cu}}{d_N} \cdot d_{\text{top}} (L) \quad L = 1, 2, 3, 4 \quad (6.12)
\]

Equation 6.13 was used to calculate the axial stress in each steel bar \( (f_s) \) assuming that the stress-strain relationship of the longitudinal steel bars is elastic-perfectly plastic in both loading conditions of tension and compression.

\[
\text{for } |\varepsilon_s| \leq \varepsilon_{s,l} \quad f_s = \varepsilon_s \cdot E_s
\]

\[
\text{for } |\varepsilon_s| > \varepsilon_{s,l} \quad f_s = f_{ys} \quad (6.13)
\]

where \( \varepsilon_{s,l} \) is yield strain of the steel bars, \( E_s \) is the modulus of elasticity of the steel bars given to be 200 GPa, and \( f_{ys} \) is the yield strength of the steel bars.
Then, the force response of each steel bar \((F_s)\) was determined by multiplying the axial stress of the steel bar by the area of the steel bar \((A_s)\) as shown in Equation (6.14).

\[
F_s = f_s \times A_s
\]  

(6.14)

The total force response of the steel bars is calculated by taking the summation of the force for all the steel bars.

The moment response in the centre of each steel bar about the centreline of the cross-section was calculated using Equation 6.15.

\[
M_s(L) = F_s(L) \times ds_{centre}(L) \\
L = 1, 2, 3, 4
\]  

(6.15)

The total moment response of the steel bars is calculated by taking the summation of the moment for all the steel bars.

### 6.3.3 Calculation of the longitudinal CFRP response

For the CFRP-confined specimen (Groups CR, CRP and CS), there is one layer of CFRP with 0.5 mm thickness oriented in the longitudinal direction as shown in Figure 6.5. To calculate the response of this CFRP layer in the longitudinal direction, a layer-by-layer numerical integration approach was adopted in similar way to that one used to calculate the response of the RPC. Each layer has a thickness of \((\Delta h)\) and a width of \((b)\) and the total number of the layers is \((n_{f,layers})\). For each layer, the position from the top and the position from the centreline are calculated by using Equation 6.16 and Equation 6.17, respectively.

\[
d_{f, top}(L) = (L - 0.5)\Delta h \\
L = 1, 2, 3, ..., n_{f,layers}
\]  

(6.16)
\[ d_{f,\text{centre}}(L) = \frac{D_0}{2} - d_{f,\text{top}}(L) \quad L = 1, 2, 3, \ldots, n_{f,\text{layers}} \] (6.17)

Figure 6.5 Layer-by-Layer division of the longitudinal CFRP

For each layer, the value of the longitudinal CFRP axial strain at the centre of each layer \( (\varepsilon_{f,\text{layer}}(L)) \) can be calculated by using Equation 6.18 below:

\[ \varepsilon_{f,\text{layer}}(L) = \varepsilon_{cu} - \frac{\varepsilon_{cu}}{d_N} d_{f,\text{top}}(L) \] (6.18)

A linear stress-strain relationship for the CFRP layer presented in Figure 5.7, Chapter 5 was used to calculate the axial stress of each layer for the longitudinal CFRP. Accordingly, the force of the longitudinal CFRP in the centre of each layer \( (F_f(L)) \) was calculated by Equation 6.19.

\[ F_f(L) = f_f \times A_f(L) \] (6.19)
where, $A_f(L)$ is the area of the CFRP layer which can calculated by multiplying the thickness of the layer $\Delta h$ by its width $b(L)$.

For the first and last layers only, the width of these two layers was calculated using Equation 6.20. Whereas, the width of the other layers in between was calculated using Equation 6.21.

$$b(L) = 2 \sqrt{\left(\frac{D_{of}}{2}\right)^2 - \left(d_{f,centre}(L)\right)^2} \quad (6.20)$$

$$b(L) = 2 \left[\sqrt{\left(\frac{D_{of}}{2}\right)^2 - \left(d_{f,centre}(L)\right)^2} - \sqrt{\left(\frac{D_{if}}{2}\right)^2 - \left(d_{f,centre}(L)\right)^2}\right] \quad (6.21)$$

Where, $D_{of}$ is the outer diameter of the longitudinal CFRP tube, $D_{if}$ is the inner diameter of the longitudinal CFRP tube, $L = 1, 2, 3, ..., n_{f,\text{layers}}$.

The total force response of the longitudinal CFRP is calculated by taking the summation of the force for all the longitudinal CFRP layers.

The moment response in the centre of each layer about the centreline of the cross-section was calculated using Equation 6.22.

$$M_f(L) = f_f(L) \times d_{f,centre}(L) \quad L = 1, 2, 3, ..., n_{f,\text{layers}} \quad (6.22)$$

The total moment response of the longitudinal CFRP cross-section is calculated by taking the summation of the moment for the longitudinal CFRP layers above the neutral axis.
6.3.4 Calculation of internal tube response

For specimens in Groups CRP and CS, internal tubes of PVC and steel were used, respectively. These internal tubes had a thickness of 3.5 mm, as presented in the experimental program of Chapter 5. To calculate the response of the PVC and steel tubes, a layer-by-layer numerical integration approach was adopted in similar way to that one used to calculate the response of the CFRP tube in the longitudinal direction. Each layer has a thickness of \((\Delta h)\) and a width of \((b)\) and the total number of the layers is \((n_{t,\text{layers}})\), as can be seen in Figure 6.6. For each layer, the position from the top and the position from the centreline are calculated by using Equation 6.23 and Equation 6.24, respectively.

\[
d_{t,\text{top}}(L) = (L - 0.5)\Delta h \quad L = 1, 2, 3, ..., n_{t,\text{layers}} \tag{6.23}
\]

\[
d_{t,\text{centre}}(L) = \frac{D_0}{2} - d_{t,\text{top}}(L) \quad L = 1, 2, 3, ..., n_{t,\text{layers}} \tag{6.24}
\]

Figure 6.6 Layer-by-Layer division of the PVC or steel internal tube
For each layer, the value of the internal tube (PVC or steel) axial strain at the centre of each layer ($\varepsilon_{t,layer}(L)$) can be calculated by using Equation 6.25 below:

$$\varepsilon_{t,layer}(L) = \varepsilon_{cu} - \frac{\varepsilon_{cu}}{d_N} d_{t,\text{top}}(L)$$ (6.25)

Equation 6.26 was used to calculate the axial stress in each layer of the internal tube, assuming that the stress-strain relationship of these tubes is elastic-perfectly plastic in both loading conditions of tension and compression.

$$\begin{align*}
\text{for } |\varepsilon_t| \leq \varepsilon_{t,l} & \quad f_t = \varepsilon_t \cdot E_t \\
\text{for } |\varepsilon_t| > \varepsilon_{t,l} & \quad f_t = f_{y,t}
\end{align*}$$ (6.26)

where, $\varepsilon_t$ is yield strain of the internal tube, $E_t$ is the modulus of elasticity of the internal tube given to be 4 GPa and 200 GPa for the PVC and steel, respectively, and $f_{y,t}$ is the yield strength of the internal tube.

The properties of the PVC tube in tension and compression presented in Section 5.3.4 of Chapter 5 was used in the calculation of Equation 6.26. Whereas, the properties of the steel tube in tension and compression presented in Section 5.3.5 of Chapter 5 was also adopted in the calculation of Equation 6.26.

Accordingly, the force of the internal tube in the centre of each layer ($F_t(L)$) was calculated by Equation 6.27.

$$F_t(L) = f_t \times A_t(L)$$ (6.27)

where, $A_t(L)$ is the area of the steel tube layer which can calculated by multiplying the thickness of the layer $\Delta h$ by its width $b(L)$. 

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For the first and the last seven layers of the internal tube only, the width of these layers was calculated using Equation 6.28. Whereas, the width of the other layers in between was calculated using Equation 6.29.

\[ b (L) = 2 \sqrt{\left( \frac{D_{ot}}{2} \right)^2 - \left( d_{t,centre}(L) \right)^2} \]  

\[ b (L) = 2 \left[ \sqrt{\left( \frac{D_{ot}}{2} \right)^2 - \left( d_{t,centre}(L) \right)^2} - \sqrt{\left( \frac{D_{it}}{2} \right)^2 - \left( d_{t,centre}(L) \right)^2} \right] \]  

where, \( D_{ot} \) is the outer diameter of the internal tube, \( D_{it} \) is the inner diameter of the internal tube, \( L = 1, 2, 3, \ldots, n_{t,\text{layers}} \).

The total force response of the internal tube is calculated by taking the summation of the force for all the internal tube layers.

The moment response in the centre of each layer about the centreline of the cross-section was calculated using Equation 6.30.

\[ M_t(L) = f_t(L) \times d_{t,centre}(L) \quad L = 1, 2, 3, \ldots, n_{t,\text{layers}} \]  

The total moment response of the internal tube cross-section is calculated by taking the summation of the moment for all the internal tube layers.

6.4 Procedure of the numerical integration method

In order to calculate the \( P \cdot M \) interaction diagram for each group of the HCRPC specimens, an MS-Excel spreadsheet was prepared for this purpose. The geometry of the cross-section and the properties of the materials were entered to the spreadsheet of each group of specimens manually by the user. The applied axial load (\( P \)) was assumed
to increase from zero to the maximum axial load \( P_{\text{max}} \) with a constant increment of \( P_{\text{max}}/50 \), as shown in Figure 6.7. This axial load increment was adopted so that a maximum error of 2% of the \( P_{\text{max}} \) can be achieved at the last axial load calculation which is a marginal error compared to the axial load capacity of the specimen. An initial value of 0.5 mm was assumed to the \( d_N \) and iterated with an increment of 0.25 mm to achieve a total force response of the cross-section \( (P_R) \) within 10 kN of the applied axial load \( P \), as can be seen in Figure 6.7. In high levels of the axial load, the difference of 10 kN can be adopted in order to obtain precise calculation of the cross-section response to the applied axial load. For a given geometry and materials properties, the ultimate strength and the corresponding strain of the RPC was calculated using unconfined concrete model for Group R and two different confined concrete models for the other groups, as presented above in Section 6.3.1. Figure 6.7 shows the calculation procedure of the numerical integration method used in this study.

For each MS-Excel spreadsheet, the results of each \( P-M \) interaction diagram were determined and printed in the same spreadsheet. Then, the envelope curve of each \( P-M \) diagram was plotted by the MS-Excel drawing tool.
Figure 6.7 Procedure of the theoretical $P$-$M$ calculations

START

Enter geometry of cross-section

Enter properties of materials

Enter $P = 0$ kN

$\varepsilon_c = 0.0001$

$d_N = 0.5$ mm

Calculate $\varepsilon_s, \varepsilon_f, \varepsilon_t$

Calculate $f'_c, f_k, f_f, f_t$

Calculate the total response of the cross-section $P_R, M_R$

$\varepsilon_c = \varepsilon_c + 0.0001$

IF $|P - P_R| \leq 10$ kN

NO

IF $\varepsilon_c = \varepsilon_{cu}$

NO

YES

Output $P_R, M_R$

IF $P = P_{\text{max}}$

NO

YES

END
6.5 Axial load-bending moment diagrams

For a column’s cross-section, the $P$-$M$ interaction diagram shows the maximum axial load and the corresponding bending moment can be applied on that cross-section. This means any loading combination of axial load and bending moment outside the $P$-$M$ envelope is not accepted. In this chapter, experimental and analytical procedures were adopted to create $P$-$M$ interaction diagrams for each group of the HCRPC specimens.

6.5.1 Experimental axial load-bending moment interaction diagrams

The experimental $P$-$M$ interaction diagrams of Groups R, CR, CRP and CS were created based on the test results of specimens tested under loading conditions of concentric, 25 mm eccentricity, 50 mm eccentricity and four-point bending, as presented in Chapter 5. Each experimental $P$-$M$ curve was constructed using four points, including pure bending moment point. The maximum axial load was identified as the highest value of axial load carried by the specimen before the rupture of CFRP tube is reached. The corresponding bending moment at the maximum axial load consists of primary and secondary moments. The primary moment caused by the eccentricity of the applied load, whereas the secondary moment was caused by the lateral deformation corresponding to the maximum axial load.

For specimens tested under concentric loading condition, the value of the corresponding bending moment ($M$) is zero. For specimens tested under 25 mm and 50 mm eccentricity, the value of the corresponding bending moment ($M$) was calculated using Equation 6.31.
\[ M = M_1 + M_2 = P \cdot e_{\text{exp}} + P \cdot \delta \]  \hspace{1cm} (6.31)

where, \( M_1, M_2 \) are the primary and secondary bending moments, respectively; \( P \) is the applied axial load; \( e_{\text{exp}} \) is the experimental eccentricity of loading; \( \delta \) is the lateral deformation corresponding to the maximum axial load.

For specimens tested under four-point bending, Equation 6.32 was used to calculate the value of the bending moment.

\[ M = \frac{P \cdot L}{6} \]  \hspace{1cm} (6.32)

where, \( L \) is the clear span length of the specimens under four-point bending which was 700 mm in this study.

The experimental \( P-M \) interaction diagrams of Groups R, CR, CRP and CS are shown in Figure 6.8. For concentrically loaded specimens, Groups CR, CRP and CS carried an axial load of 12.5\%, 24.5\% and 12\%, respectively larger than the axial load of Group R. By increasing the eccentricity to 25 mm, Groups CR, CRP and CS resisted an axial load of 7.6\%, 13.3\% and 11.1\% higher than the axial load of Group R and the bending moment of Groups CR, CRP and CS also increased by 21.3\%, 29.4\% and 26.9\%, respectively compared to the bending moment of Group R.
For HCRPC specimens that subjected to 50 mm eccentricity, Groups CR, CRP and CS showed 6.6%, 10.8% and 2.3% higher axial load than axial load of Group R, respectively and the corresponding bending moments were increased by 3.6%, 10.8% and 3.8%, respectively compared to the bending moment of Group R. It should be mentioned that Group R showed a higher bending moment than the actual one under 50 mm eccentric load because of an overestimation of the secondary moment ($M_2$). The reason behind this misleading calculation is that the lateral deformation reading (δ) of the laser triangulation device was taken from fully cracked concrete cover instead of the surface of the concrete cover, as shown in Figure 5.28 in Chapter 5.

For specimens tested under four-point bending, Groups CR, CRP and CS resisted bending moment of 133%, 138% and 78% larger than the bending moment of Group R, respectively.
Under four-point bending test, it can be seen from the results in Figure 4 that Groups CR and CS showed different values of bending moment, although they had been designed to resist the same axial load. The reason behind that is lack of the bond between the RPC and the internal steel tube that reduce the transferred load from the RPC to the steel tube. In addition, in the design of Group CR, the longitudinal steel bars are located to obtain higher bending moment than Group CS that had the internal steel tube located in the centre of the specimen’s cross-section.

In general, the test results presented in Figure 4 clearly shows that Group CRP exhibited larger capacity of axial load-bending moment interaction diagram than the other groups in this study.

6.5.2 Analytical axial load-bending moment interaction diagrams

This section presents a comparison between the analytical and the experimental $P-M$ interaction diagrams for all the HCRPC groups. Then, a parametric study is carried out based on the analytical results to examine the effect of using external CFRP tubes and internal PVC or steel tubes on the $P-M$ interaction diagrams. For column specimens tested under eccentric loading, the effect of both primary and secondary bending moments has been considered in the analytical $P-M$ interaction diagrams.

6.5.2.1 Axial load-bending moment interaction diagram of Group R

For unconfined HCRPC specimens (Group R), the analytical $P-M$ interaction diagram was constructed using a stress-strain concrete model proposed by Yang et al. (2014). This model was created to be applicable to unconfined concrete with compressive
strength between 10 MPa to 180 MPa. Figure 6.9 shows a comparison between the analytical and the experimental $P-M$ interaction diagrams. For Specimen R0, the predicted value of the axial load was 89.5% of the observed value. For Specimen R25, the predicted values of the axial load and bending moment were 95.6% and 86.2% of the observed values, respectively. For Specimen R50, the predicted values of the axial load and bending moment were 89% and 73.5% of the observed values, respectively. As mentioned above, the observed bending moment of Specimen R50 showed a higher value than the predicted one because of a misleading calculation of the lateral deformation reading ($\delta$). Because the reading of the laser triangulation device was taken from fully cracked concrete cover instead of the surface of the concrete cover, as a result a higher value of $M_2$ was calculated than the actual one. The experimental $P-M_1$ interaction diagram presented in Figure 6.9 below, neglects the effect of the misleading reading of the secondary bending moment in Group R. For Specimen RB, the predicted value of the bending moment was 74.7% of the observed value.

![Figure 6.9 Analytical and experimental $P-M$ interaction diagrams of Group R](image-url)
6.5.2.2 Axial load-bending moment interaction diagrams of Group CR

For CFRP-confined HCRPC specimens (Group CR), the analytical $P$-$M$ interaction diagram was constructed using a stress-strain concrete model proposed by Yazici and Hadi (2012). This model was created to be applicable to FRP-confined solid and hollow core concrete. Figure 6.10 shows a comparison between the analytical and the experimental $P$-$M$ interaction diagrams. For Specimen CR0, the predicted value of the axial load was 83.3% of the observed value. For Specimen CR25, the predicted values of the axial load and bending moment were 88.1% and 86.6% of the observed values, respectively. For Specimen CR50, the predicted values of the axial load and bending moment were 90.7% and 91.3% of the observed values, respectively. For Specimen CRB, the predicted value of the bending moment was 91.9% of the observed value.

![Figure 6.10 Analytical and experimental $P$-$M$ interaction diagrams of Group CR](image)

Figure 6.10 Analytical and experimental $P$-$M$ interaction diagrams of Group CR
6.5.2.3 Axial load-bending moment interaction diagrams of Group CRP

For CFRP-confined HCRPC specimens with an internal PVC tube (Group CRP), the analytical axial load-bending moment interaction diagram was developed using a stress-strain concrete model proposed by Xiao et al. (2010). This model was created to be suitable for FRP-confined high strength concrete with compressive strength between 60 MPa to 126 MPa. Figure 6.11 shows a comparison between the analytical and the experimental P-M interaction diagrams.

For Specimen CRP0, the predicted value of the axial load was 87.4% of the observed value. For Specimen CRP25, the predicted values of the axial load and bending moment were 93.7% and 92.3% of the observed values, respectively. For Specimen CRP50, the predicted values of the axial load and bending moment were 88.6% and 89.6% of the observed values, respectively. For specimen tested under pure bending moment

Figure 6.11 Analytical and experimental P-M interaction diagrams of Group CRP

For Specimen CRP0, the predicted value of the axial load was 87.4% of the observed value. For Specimen CRP25, the predicted values of the axial load and bending moment were 93.7% and 92.3% of the observed values, respectively. For Specimen CRP50, the predicted values of the axial load and bending moment were 88.6% and 89.6% of the observed values, respectively. For specimen tested under pure bending moment
(Specimen CRPB), the predicted value of the bending moment was 94% of the observed value.

6.5.2.4 Axial load-bending moment interaction diagrams of Group CS

For CFRP-confined HCRPC specimens with an internal steel tube (Group CS), the analytical axial load-bending moment interaction diagram was constructed using the same stress-strain concrete model that used in the calculations of $P-M$ interaction diagram of Group CRP. Figure 6.12 shows a comparison between the analytical and the experimental $P-M$ interaction diagram.

![Figure 6.12 Analytical and experimental $P-M$ interaction diagrams of Group CS](image)

For Specimen CS0, the predicted value of the axial load was 98.2% of the observed value. For Specimen CS25, the predicted values of the axial load and bending moment
were 95.5% and 90.8% of the observed values, respectively. For Specimen CS50, the predicted values of the axial load and bending moment were 92% and 93.5% of the observed values, respectively. For specimen tested under four-point bending (Specimen CSB), the predicted value of the bending moment was 15.2% higher than the observed value. This overestimation could be attributed to the lack of the bond between the RPC and the internal steel tube which reduce the transferred load from the RPC to the steel tube.

6.6 Parametric study

6.6.1 Effect of the external CFRP tube on the $P-M$ interaction diagram

The effect of using external CFRP tube confinement on the analytical $P-M$ interaction diagram of HCRPC specimens can be examined by comparing the analytical $P-M$ interaction diagrams of Group R and Group CR, as shown in Figure 6.13. From the analytical results presented in Figure 6.13 it was found that for specimen tested under concentric load, the axial load was increased by 6.6% due to the effect of using CFRP confinement. This small increase could be attributed to the effect of the hollow core that minimizes the effect of CFRP confinement and this behaviour was reported by previous studies such as Fam and Rizkalla (2001) and Wong et al. (2008).

For specimens tested under 25 mm eccentric load, the axial load of Specimen CR25 was 14.2% higher than the axial load of Specimen R25. Also, the bending moment of Specimen CR25 was 21.9% higher than the bending moment of Specimen R25. For specimens tested under 50 mm eccentric load, the axial load and the bending moment of Specimen CR50 was 22.1% and 14.9% higher than the axial load and the bending moment of Specimen R50, respectively.
For specimens tested under four-point bending, a significant bending moment increase was observed as Specimen CRB showed a bending moment of 187% higher than Specimen RB due to the effect of the CFRP layer in the longitudinal direction of the specimen. Accordingly, the main advantage of using CFRP tube with longitudinal FRP layer in HCRPC specimen is to enhance the bending moment capacity.

Figure 6.13 Analytical $P$-$M$ interaction diagrams of Group R and Group CR

6.6.2 Effect of the internal PVC tube on the $P$-$M$ interaction diagram

In order to investigate the effect of using an internal PVC tube on the analytical $P$-$M$ interaction diagram of HCRPC specimens, the analytical $P$-$M$ interaction diagrams of Group CR and Group CRP were compared with each other, as shown in Figure 6.14. It can be clearly seen from the analytical results presented in Figure 6.14 that the using of an internal PVC tube had an important effect on the analytical $P$-$M$ interaction diagrams. By using an internal PVC tube within the CFRP-confined HCRPC specimens,
the RPC became under tri-axial confinement and different model of stress-strain was used to calculate the $P$-$M$ interaction diagrams, as mentioned above.

![Analytical $P$-$M$ interaction diagrams of Group CR and Group CRP](image)

Figure 6.14 Analytical $P$-$M$ interaction diagrams of Group CR and Group CRP

For concentrically loaded specimens, the axial load of Specimen CRP0 was 16.1% higher than the axial load of Specimen CR0. For specimens tested under 25 mm eccentric load, the axial load and bending moment of Specimen CRP25 was 12% higher than the axial load of Specimen CR25, whereas, the bending moment of Specimen CRP25 was 14.8% lower than the bending moment of Specimen CR25. For specimen tested under 50 mm eccentric load, the axial load of Specimen CRP50 was 1.4% higher than the axial load of Specimen CR50, while, the bending moment of Specimen CRP50 was 9.5% lower than the bending moment of Specimen CR50. For specimens tested under four-point bending, Specimen CRPB showed a bending moment of 4.6% higher than Specimen CRB. Accordingly, using of internal PVC tubes in CFRP-confined HCRPC specimens has the advantages of enhancing the axial load carrying capacity, in addition to the low cost and light self-weight material.
6.6.3 Effect of replacing steel reinforcement with steel tube on the P-M interaction diagram

The effect of replacing normal steel reinforcement with steel tube on the analytical axial load-bending moment diagrams of CFRP-confined HCRPC specimens can be examined by comparing the analytical P-M interaction diagrams of Group CR and Group CS, as shown in Figure 6.15. Based on the analytical results presented in Figure 6.15, for concentric loaded specimens, the axial load of Specimen CS0 showed nearly the same axial load of Specimen CR0.

For specimens tested under 25 mm eccentric load, the axial load and bending moment of Specimen CS25 was 1.4% higher than the axial load of Specimen CR25, whereas, the bending moment of Specimen CS25 was 18% lower than the bending moment of Specimen CR25. For specimen tested under 50 mm eccentric load, the axial load of Specimen CS50 was 7.5% higher than the axial load of Specimen CR50, while, the bending moment of Specimen CS50 was 15.3% lower than the bending moment of Specimen CR50. For specimens tested under pure bending moment, Specimen CSB showed a bending moment of 7.6% lower than Specimen CRB.
Based on the results presented above, the replacing of normal steel reinforcement with an internal steel tube in HCRPC specimens increases the axial load capacity of this type of specimens but it also reduce the bending moment capacity.

### 6.6.4 Effect of the hollow core size on the P-M interaction diagram

The effect of the hollow core size on the \( P-M \) interaction diagram was investigated using analytical method of the numerical integration presented above. Three different diameters of the hollow core \( (D_i) \) were used (60 mm, 90 mm and 120 mm) to examine this effect. Figure 6.16 shows the \( P-M \) interaction diagrams of the HCRPC specimens with internal hollow core diameter of 60 mm, 90 mm and 120 mm. Based on the results presented in Figure 6.16, it can be noticed that increasing \( D_i \) led to decreasing the load capacity of the \( P-M \) interaction diagrams in the HCRPC specimens. For HCRPC specimens under concentric loading condition, the maximum axial load of the HCRPC
specimens of $D_i = 90$ mm and $D_i = 60$ mm were decreased by 87.5% and 74%, respectively compared to the maximum axial load of HCRPC specimen of $D_i = 90$ mm. It was also observed that changing the hollow core size has minor effect on the $P$-$M$ interaction diagrams of the HCRPC specimens with increasing load eccentricity, especially under pure bending loading condition, as can be seen in Figure 6.16. This could be due to the fact the hollow core was located away from the tension side of the HCRPC cross-section.

![Figure 6.16 Effect of $D_i$ on the $P$-$M$ interaction diagrams of HCRPC specimens](image)

**Figure 6.16 Effect of $D_i$ on the $P$-$M$ interaction diagrams of HCRPC specimens**

### 6.6.5 Effect of the longitudinal reinforcement ratio on the P-M interaction diagram

It is well-known that increasing the longitudinal steel reinforcement ratio in solid concrete columns can significantly increase the axial load capacity. However, the effect of the longitudinal steel reinforcement ratio in hollow core concrete columns could be less than solid ones, due to the effect of the hollow core. The effect of the longitudinal
reinforcement ratio on the $P$-$M$ interaction diagrams of the HCRPC specimens is shown in Figure 6.17. Four different diameters of the longitudinal reinforcement ratio ($\rho$) were used (1.7%, 2.5%, 3.4% and 4.5%) to examine this effect. Based on the results presented in Figure 6.17, it can be noticed that increasing $\rho$ led to decreasing the load capacity of the $P$-$M$ interaction diagrams in the HCRPC specimens. For HCRPC specimens under concentric loading condition, the maximum axial load of the HCRPC specimens of $\rho = 2.5\%$, $\rho = 3.4\%$ and $\rho = 4.5\%$ mm were increased by 5%, 11% and 17%, respectively compared to the maximum axial load of HCRPC specimen of $\rho = 1.7\%$. It was also observed that increasing the longitudinal steel reinforcement ratio has significant effect on the $P$-$M$ interaction diagrams of the HCRPC specimens with increasing load eccentricity, especially under pure bending loading condition, as can be seen in Figure 6.17. The bending moment of the HCRPC specimens of $\rho = 2.5\%$, $\rho = 3.4\%$ and $\rho = 4.5\%$ mm were increased by 46%, 91% and 144%, respectively compared to the bending moment of HCRPC specimen of $\rho = 1.7\%$.

![Figure 6.17](image_url)
6.7 Summary

This chapter presented the analytical results of the $P$-$M$ interaction diagrams of the all HCRPC specimens based on a layer-by-layer numerical integration approach. According to the analytical results shown in this chapter, the $P$-$M$ interaction diagrams of the HCRPC specimens can be modelled with an acceptable accuracy by using existing stress-strain models of both unconfined and FRP-confined concrete. In general, the analytical $P$-$M$ interaction diagrams showed conservative predictions to the experimental $P$-$M$ interaction diagrams. For CFRP-confined specimens, the underestimated prediction was due to the fact that the analytical results of the axial load and bending moment were calculated based on the ultimate strain of FRP-confined concrete in compression. In addition, for CFRP-confined specimens tested under eccentric load, the actual strain of FRP-confined concrete can be higher than the analytical strain of FRP-confined concrete in the most compressed concrete area.

The effect of an external CFRP tube confinement, an internal PVC tube and replacing the normal steel reinforcement with an internal steel tube were investigated based on the analytical results. It was found that the using of CFRP layer in the longitudinal direction had a significant effect to increase the bending moment capacity of HCRPC specimen. In addition, the using of an internal PVC tube in CFRP-confined HCRPC specimens has the advantages of enhancing the axial load carrying capacity. Whereas, replacing the normal steel reinforcement with internal steel tube in HCRPC specimens increases the axial load capacity of this type of specimens but it also reduces the bending moment capacity.
The next chapter presents summarised conclusions based on the experimental and analytical studies conducted in this study. Recommendations for future studies are also suggested about HCRPC specimens.
7 CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions of the study
The main purpose of this study was to investigate the behaviour of CFRP-confined HCRPC column with an internal PVC tube. This study was designed to obtain more knowledge of this new type of hollow core concrete column. A total of 16 circular HCRPC short concrete specimens, divided into four groups were prepared and tested under different loading conditions. The experimental program of this study aimed to investigate the effect of different parameters on the behaviour of HCRPC specimens, such as external CFRP tube confinement, using an internal PVC tube and replacing normal steel reinforcement with an internal steel tube. Analytical study was also conducted to predict the axial load-bending moment interaction diagrams of the HCRPC specimens using numerical integration approach.

Based on the experimental and analytical results presented in this study, the following conclusions can be drawn:

1. The tensile strength of the RPC can be enhanced by increasing the volume fraction of the steel fibres within the RPC mix and the tensile strain hardening can be achieved with 3% of steel fibre by volume of the RPC.

2. For the RPC, the splitting test was overestimating the tensile strength. In addition, by increasing the steel fibre content, the overestimation of the tensile strength was increased. Also, The Double Punch Test (DPT) showed more accurate tensile strength of the RPC than the splitting test when compared with the Direct Tensile Test (DTT).
3. The CFRP-confined hollow core specimens with an inner PVC tube generally possessed a good ductility and were higher than their counterparts without a PVC tube. This was due to the beneficial effect of the PVC tube which provided constraints/confine ment from the inside.

4. Under the same axial strain, the outward lateral expansion of CFRP-confined hollow core specimens was generally lower than the corresponding solid specimens. This suggests that the ultimate axial strain of the former may be larger than the latter for the same confining material.

5. Compared with hollow core specimens without an inner tube, the presence of an inner PVC tube led to an increased outward expansion of the CFRP-confined specimens, but this effect was only obvious when the CFRP confinement was strong (i.e. by using a two-layer wrap).

6. For unconfined specimens, solid specimens exhibited higher ductility than hollow specimens. For confined specimens, however, the ductility of hollow specimens with an internal PVC tube can be enhanced to show close values of ductility compare to those of the solid specimens.

7. The strength of HCRPC specimens was slightly increased by using CFRP tube confinement and the ductility was significantly improved when confinement of CFRP tube was introduced.

8. By increasing the eccentricity of loading, the axial load capacity of all CFRP-confined HCRPC specimens having different configurations decreased in comparison with the unconfined HCRPC specimen. On the other hand, the axial deformation capacity of all CFRP-confined specimens was observed to increase dramatically with the increase of loading eccentricity. Thus, higher values of
ductility were achieved by the 50-mm eccentric loaded specimens compared to those specimens tested under 25-mm eccentricity.

9. The four-point bending test indicates that the use of a CFRP tube can significantly increase the maximum load and ductility of HCRPC specimens.

10. By replacing the conventional steel reinforcement with an equivalent steel tube within the section of HCRPC specimens, the values of strength and ductility are nearly the same. However, under flexural loading a better performance of HCRPC specimen with normal steel reinforcement can be achieved than the one with the steel tube.

11. By providing an inner PVC tube to the hollow core RPC specimens, which are internally reinforced with conventional steel reinforcement and externally confined with a CFRP tube, the strength and ductility are improved. This is because of the beneficial effect of the PVC tube that provides an additional inner confinement to the annular concrete section.

12. The axial load-bending moment interaction diagrams of the HCRPC specimens can be modelled with an acceptable accuracy by using existing stress-strain models of both unconfined and CFRP-confined concrete.
7.2 Recommendations for future studies

Further research options are possible on the following key areas:

1. Using different types of FRP tube such as Glass FRP (GFRP) and Aramid FRP (AFRP) as an external confinement for the HCRPC column. Different types of FRP tubes are expected to show different confinement behaviour in terms of strength and ductility. The CFRP tubes used in this study for confining HCRPC slightly increased the axial load capacity of the specimens due to the effect of the hollow core. For this reason, it might be a good option to use a low cost FRP tubes (e.g. GFRP tubes) to confine HCRPC specimens. Normally, the GFRP tubes have lower tensile strength, lower modulus of elasticity and higher tensile strain than the CFRP tube.

2. Replacing normal steel reinforcement (steel bars and helices) with FRP reinforcement to provide the HCRPC columns with more resistance to the corrosion effect and the design life of the structure can be increased in this case. In addition, using hollow core column sections is mainly to reduce the total self-weight of the structure. Thus, by replacing normal steel reinforcement with FRP reinforcement, more reduction can be obtained in the total self-weight of the structure.

3. Continue with similar research study on FRP-confined hollow core columns with different cross-section such as square or rectangular cross-sections. The size and the shape of the hollow core in HCRPC specimens can also be investigated in order to obtain a reliable research background for a design-guideline.
References

ACI (American Concrete Institute). (2008). “Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures.” 440.2R-08, Farmington Hills, MI 48331, USA.


# Appendix A: Summary of selected FRP-confined concrete models

## Table A1 Confinement models

<table>
<thead>
<tr>
<th>Model</th>
<th>Ultimate condition equations</th>
<th>Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Strength</td>
<td>Strain</td>
</tr>
<tr>
<td>Richart et al. (1928)</td>
<td>( \frac{f'<em>{cc}}{f'</em>{co}} = 1 + 4.1 \frac{f_i}{f'_{co}} )</td>
<td>( \frac{\varepsilon_{cc}}{\varepsilon_{co}} = 1 + 5 \left( \frac{f'<em>{cc}}{f'</em>{co}} - 1 \right) )</td>
</tr>
<tr>
<td>Fardis &amp; Khalili (1981)</td>
<td>( \frac{f'<em>{cc}}{f'</em>{co}} = 1 + 3.7 \left( \frac{f_i}{f'_{co}} \right)^{0.86} )</td>
<td>( \varepsilon_{cc} = \varepsilon_{co} + 0.0005 \left( \frac{E_l}{f'_{co}} \right) )</td>
</tr>
<tr>
<td></td>
<td>Where, ( E_l ), Lateral confinement stiffness = ( \frac{2E_{ltf}}{D_o} ) (MPa)</td>
<td></td>
</tr>
<tr>
<td>Mander et al. (1988)</td>
<td>( \frac{f'<em>{cc}}{f'</em>{co}} = 2.254 \sqrt{1 + 7.94 \frac{f_i}{f'<em>{co}} - 2 \frac{f_i}{f'</em>{co}} - 1.254} )</td>
<td>-----</td>
</tr>
<tr>
<td>Miyauchi et al. (1997)</td>
<td>( \frac{f'<em>{cc}}{f'</em>{co}} = 1 + 3.485 \left( \frac{f_i}{f'_{co}} \right) )</td>
<td>( \frac{\varepsilon_{cc}}{\varepsilon_{co}} = 1 + 10.6 \left( \frac{f_i}{f'<em>{co}} \right)^{0.373} ) ( f'</em>{co} = 30 \text{ MPa} )</td>
</tr>
<tr>
<td></td>
<td>( \varepsilon_{cc} = 1 + 10.5 \left( \frac{f_i}{f'<em>{co}} \right)^{0.525} ) ( f'</em>{co} = 50 \text{ MPa} )</td>
<td></td>
</tr>
<tr>
<td>Toutanji (1999)</td>
<td>( \frac{f'<em>{cc}}{f'</em>{co}} = 1 + 3.5 \left( \frac{f_i}{f'_{co}} \right)^{0.85} )</td>
<td>( \frac{\varepsilon_{cc}}{\varepsilon_{co}} = 1 + (310.57\varepsilon_{fu} + 1.90) \left( \frac{f'<em>{cc}}{f'</em>{co}} - 1 \right) )</td>
</tr>
</tbody>
</table>
Table A1 (Contd.)

<table>
<thead>
<tr>
<th>Lam and Teng (2003)</th>
<th>$f'_{cc} = f'_c \left(1 + 2 \left(\frac{f_t + f_i}{f'_c}\right)\right)$</th>
<th>$\varepsilon_{cc} = \varepsilon_{co} (1.75 + 12 \left(\frac{f_i}{f'<em>c}\right)^{\frac{\varepsilon</em>{h,rup}}{\varepsilon_{co}}}^{0.45})$</th>
</tr>
</thead>
</table>
| Wu et al. (2006)    | Hardening behaviour  
(For normal modulus FRP)  
$\frac{f'_{cc}}{f'_{co}} = 2 \left(\frac{f_t}{f'_{co}}\right) + 1$  
(For High modulus FRP)  
$\frac{f'_{cc}}{f'_{co}} = 2.4 \left(\frac{f_t}{f'_{co}}\right) + 1$ | (For normal modulus FRP)  
$\varepsilon_{cc} = 1.785 \varepsilon_{fu} \left(\frac{f_i}{f'_{co}}\right)^{1.515}$  
(For High modulus FRP)  
$\varepsilon_{cc} = 1.785 \varepsilon_{fu} \left(\frac{f_i}{k_1 f'_{co}}\right)^{1.515}$  
Where $k_1 = \sqrt{250/E_f}$ (E in GPa and $\geq 250$) |
|                     | Softening behaviour  
$f'_{cc} = f'_{co} \left(1 + 0.002 \frac{30}{f'_{co}} \frac{\rho f E_f}{\sqrt{f'_{cc}}}\right)$  
$f'_{cc} = f'_{co} \left(0.75 + 2.5 \frac{f_t}{f'_{co}}\right)$ | $\varepsilon_{cc} = \varepsilon_{co} \left(1 + 0.007 \frac{30}{f'_{co}} \frac{\rho f E_f}{\sqrt{f'_{cc}}}\right)$  
$\varepsilon_{cc} = \varepsilon_{co} \left(1.3 + 6.3 \frac{f_t}{f'_{co}}\right)$ |
| Youssef et al. (2007)| $\frac{f'_{cc}}{f'_{co}} = 1 + 2.25 \left(\frac{f_t}{f'_{co}}\right)^{\frac{5}{4}}$ | $\varepsilon_{cc} = 0.003368 + 0.259 \left(\frac{f_t}{f'_{co}}\right) \left(\frac{f_f}{E_f}\right)$ |
Table A1 (Contd.)

<table>
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<th>Scenario</th>
<th>Expression</th>
<th>Parameterization</th>
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<tr>
<td>Jiang and Teng (2007)</td>
<td>$\frac{f'<em>{cc}}{f'</em>{co}} = 1 + 2.25 \left( \frac{f_{t}}{f'_{co}} \right)^{0.4}$</td>
<td>$\varepsilon_{cc} = 0.003368 + 0.259 \left( \frac{f_{t}}{f'<em>{co}} \right) \left( \frac{f</em>{f}}{E_{f}} \right)$</td>
</tr>
<tr>
<td>Yu et al. (2010)</td>
<td>$\frac{f'<em>{cc}}{f'</em>{co}} = \begin{cases} 1 + 3.5(\rho_{K} - 0.01)\rho_{e} &amp; \text{if } \rho_{K} \geq 0.01 \ 1 &amp; \text{if } \rho_{K} &lt; 0.01 \end{cases}$</td>
<td>$\frac{\varepsilon_{cc}}{\varepsilon_{co}} = 1.75 + 6.5 \rho_{K}^{0.8} \rho_{e}^{1.45} (1 - \varphi)^{-0.22}$</td>
</tr>
<tr>
<td></td>
<td>$\rho_{K} = \frac{E_{f} t_{f}}{(E_{sec} R_{0})}$</td>
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<tr>
<td></td>
<td>$\rho_{e} = \frac{\varepsilon_{h,rup}}{\varepsilon_{co}}$</td>
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<tr>
<td></td>
<td>$E_{sec} = f'<em>{co} / \left( \varepsilon</em>{co} f_{t} \right)$</td>
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<tr>
<td>Yazici and Hadi (2012)</td>
<td>$\frac{f'<em>{cc}}{f'</em>{co}} = (1 + 0.033 K_{N})\beta$</td>
<td>$\frac{\varepsilon_{cc}}{\varepsilon_{co}} = (1 + 0.16 K_{N})\beta$</td>
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<td>$K_{N} = \frac{2 E_{f} t_{f}}{D_{o} f'_{co}}$</td>
<td>$10 \leq K_{N} \leq 20$</td>
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<td>$\beta = \left( 1 - \frac{D_{i}^{2}}{D_{o}^{2}} \right)$</td>
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### Appendix B: Trail mix designs of RPC

#### Table B1 Trial mixes of RPC

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<tr>
<td>GP Cement (kg/m(^3))</td>
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<td>800</td>
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<td>Silica Fume (kg/m(^3))</td>
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<td>236</td>
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<td>Fine Sand (kg/m(^3))</td>
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<td>1036</td>
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<td>Superplasticizer (SP) (kg/m(^3))</td>
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<td>Steel Fibre (kg/m(^3))</td>
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<td>Water (w) (kg/m(^3))</td>
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<td>Fly ash (kg/m(^3))</td>
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<td>Binder (b) (kg/m(^3))</td>
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<td>w/b</td>
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<td>SP/b (%)</td>
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#### Table B2 Test Results of RPC trial mixes

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<th>Trial Mix Reference</th>
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<th>Compressive strength (MPa)</th>
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<td></td>
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<td>7 Days</td>
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<tr>
<td>1st Trail</td>
<td>Cylinder 100×200 mm</td>
<td>6</td>
<td>Mortar Mixer</td>
<td>150</td>
<td>62</td>
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<tr>
<td>2nd Trail</td>
<td>Cylinder 100×200 mm</td>
<td>6</td>
<td>Drum Mixer</td>
<td>125</td>
<td>57</td>
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<td>3rd Trail</td>
<td>Cylinder 100×200 mm</td>
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<td>Mortar Mixer</td>
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<td>4th Trail</td>
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<td>Mortar Mixer</td>
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