Behaviour of circular high strength concrete columns reinforced with micro, macro and hybrid steel fibres under different loading conditions

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Behaviour of Circular High Strength Concrete Columns Reinforced with Micro, Macro and Hybrid Steel Fibres Under Different Loading Conditions

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

DOCTOR OF PHILOSOPHY

By

Emdad Kadhim Zainel BALANJI

2017
ABSTRACT

This study investigates experimentally and analytically the behaviour of High Strength Concrete (HSC) columns reinforced with micro steel fibres, macro steel fibres and hybrid steel fibres under different loading conditions (concentric axial load, 25 mm and 50 mm eccentric axial loads and four-point bending). The influence of the type, volume content and aspect ratio (length to diameter ratio) of steel fibres on the strength and ductility of HSC columns were investigated by testing 16 circular specimens of 200 mm diameter and 800 mm height. All the specimens were reinforced with the same amount of longitudinal and helical steel reinforcement. The specimens were divided into four groups (Group RC, Group MI, Group MA and Group HY) of four specimens. Group RC specimens contained no steel fibres, Group MI specimens contained 3% by volume of micro steel fibres, Group MA specimens contained 2% by volume of macro steel fibres, and Group HY specimens contained 2.5% by volume of hybrid steel fibres (1.5% of micro steel fibres and 1% of macro steel fibres). A total of 12 specimens were tested as columns with different eccentricities (concentric axial load, 25 mm and 50 mm eccentric axial loads) and four specimens were tested as beams under four-point bending. In addition to the experimental work, analytical studies were conducted to develop axial load-bending moment interaction diagrams of Groups RC, MI, MA and HY using the equivalent rectangular stress block method and layer-by-layer integration method. Also, bending moment-curvature diagrams of the specimens tested under 25 mm and 50 mm eccentric axial load were developed using layer-by-layer integration method.
The experimental results showed that the inclusion of micro steel fibres and hybrid steel fibres into HSC (Group MI and Group HY) enhanced the strength and ductility of the specimens under eccentric axial loads. The results also showed that the addition of macro steel fibres into HSC (Group MA) enhanced the ductility but reduced the strength of the specimens compared to Group RC specimens. The analytical axial load-bending moment interaction diagrams underestimated the corresponding experimental axial load-bending moment interaction diagrams. Also, the analytical axial load-bending moment interaction diagrams constructed from the layer-by-layer method were close to the experimental axial load-bending moment interaction diagrams. In addition, the analytical bending moment-curvature diagrams were in good agreement with the corresponding experimental bending moment-curvature diagrams.
THESIS DECLARATION

I, Emdad K. Z. Balanji, 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 work unless otherwise referenced or acknowledged. The document has not been submitted for qualification at any other academic institution.

Emdad K. Z. Balanji
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Emdad K. Z. Balanji
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ABBREVIATIONS

\( A_c \) : Column core area (mm\(^2\))

\( A_e \) : Area of the effective confined columns (mm\(^2\))

\( A_{sh} \) : Cross-section of one leg of hoop

\( b_c \) : Core dimensions to the centreline of the ties across width (mm)

\( D \) : Diameter of the cross-section

\( d_c \) : Core dimensions to the centreline of the ties across depth (mm)

\( d_s \) : Diameter of the lateral reinforcement (mm)

\( d_f \) : Diameter of fibre (mm)

\( d_n \) : Depth of neutral axis of cross-section

\( d_x \) : Distance of the longitudinal steel bar from the top extreme fibre compression (mm)

\( E_c \) : Modulus of elasticity of the plain concrete in compression (GPa)

\( E_{cf} \) : Modulus of elasticity of fibre reinforced concrete (GPa)

\( E_{it} \) : Initiation slope of axial stress-strain curve of concrete in compression (GPa)

\( E_s \) : Modulus of elasticity of transverse steel (GPa)

\( E_{ot} \) : Initiation slope of axial stress-strain curve of concrete in tension (GPa)

\( E_{ct} \) : Modulus of elasticity of the plain concrete in tension

\( E_{sf} \) : Modulus of elasticity of transverse steel fibre (GPa)

\( f_c \) : Stress of the concrete

\( f'_{cf} \) : Unconfined compressive strength of the fibre reinforced concrete (MPa)
\( f_{sp}' \): Splitting tensile strength of the plain concrete (MPa)
\( f_{spf}' \): Splitting tensile strength of the steel fibre reinforced concrete (MPa)
\( f_r' \): Flexural strength of the plain concrete (MPa)
\( f_{rf}' \): Flexural strength of the fibre reinforced concrete (MPa)
\( f_{rt} \): Residual tensile stress (MPa)
\( f_{ct} \): Ultimate tensile stress (MPa)
\( f_h \): Lateral steel stress at concrete peak stress (MPa)
\( f_t \): Confinement pressure provided by the lateral reinforcement
\( f_b \): Confinement pressure provided by the steel fibres
\( f_m \): Tensile stress of the matrix (MPa)
\( f_y \): Yield stress of stirrups (MPa)
\( f_{c}' \): Unconfined compressive strength of the concrete obtains from cylinder (150 x 300 mm) (MPa)
\( f_{cc}' \): Confined compression strength of the concrete in member (MPa)
\( l_e' \): Effective confinement index evaluated at peak strength
\( I_{10} \): Ductility factors
\( k_e \): Confinement effectiveness coefficient
\( k_1 \): Coefficient that relates confinement pressure to strength enhancement
\( k_2 \): Coefficient that reflects efficiency of confinement reinforcement
\( k_3 \): Coefficient to reflect effect of concrete strength
\( k_4 \): Coefficient to reflect effect of transverse steel strength
\( K \): Coefficient of strength enhancement
\( l_f \): Length of fibre (mm)
\( \frac{l_t}{d_f} \): Aspect ratio of fibres

\( n \): Spacing between longitudinal bars (mm)

\( N_{uo} \): Nominal axial load (kN)

\( N_{uoc} \): Compressive strength of the total concrete cross-sectional area (kN)

\( N_{uocc} \): Compressive strength of the total concrete core (kN)

\( P_{max} \): Maximum axial load (kN)

\( P_{yield} \): Yield axial load (kN)

\( P_{1p} \): First peak axial load (kN)

\( P_{core} \): Second peak load (kN)

\( R. I. \): Reinforced index of fibre

\( s' \): Clear spacing between helix or hoop (mm)

\( s \): Pitch of the steel helix or hoop (mm)

\( s_l \): Spacing of longitudinal reinforcement, laterally supported by corner of hoop or hook of crosstie

\( TR \): Toughness ratio of the plain concrete

\( TR_f \): Toughness ratio of fibre reinforced concrete

\( \nu_f \): Volume content of fibres (%)

\( \beta \): Material parameter

\( \varepsilon_c \): Strain of the concrete

\( \varepsilon'_i \): Strain at 0.3 \( f'_c \) in the descending branch of the stress-strain curve.

\( \varepsilon'_{cf} \): Axial strain of the steel fibre reinforced concrete at \( f_{cf} \)

\( \varepsilon'_c \): Axial strain corresponding of unconfined concrete at \( f'_c \)

\( \varepsilon'_{cc} \): Axial strain of confined concrete at \( f'_{cc} \)

\( \varepsilon'_{c50} \): Axial strain of unconfined concrete on descending branch at 0.5\( f'_c \)
\( \varepsilon'_{50c} \) : Axial strain of confined concrete on descending branch at 0.5\( f'_{cc} \)

\( \varepsilon'_{c85} \) : Axial strain of unconfined concrete on descending branch at 0.85\( f'_c \)

\( \varepsilon'_{c85c} \) : Axial strain of confined concrete on descending branch at 0.85\( f'_{cc} \)

\( \varepsilon_{ct} \) : Tensile strain corresponding to \( f_{ct} \)

\( \varepsilon'_u \) : Ultimate strain of plain concrete

\( \varepsilon_{rt} \) : Residual tensile strain corresponding to \( f_{rt} \)

\( \sigma_{t1} \) : Tensile stress of steel fibre reinforced concrete (MPa)

\( \rho_{cc} \) : Ratio of area of longitudinal reinforcement to the area of core of the section (%)

\( \rho_c \) : Total transverse steel area in two orthogonal directions divided by corresponding concrete area (%)

\( \rho_s \) : Volume of lateral steel divided by the volume of core concrete (%)

\( \rho_{sey} \) : Effective volumetric lateral reinforcement ratio

\( \Delta_1 \) : The yield deflection (mm)

\( \Delta_{80} \) : Axial deformation of 80% of the maximum axial load (mm)

\( \mu_{80} \) : Ductility based on 80% of maximum axial load

\( \eta_0 \) : Fibre length factor

\( \eta_\theta \) : Fibre orientation factor

\( \eta_{50} \) : Factor that considers the influence of the fibres when the axial strain is equal to the post-peak strain

\( \tau_{fu} \) : Bond shear strength of fibre (MPa)

\( \phi \) : Curvature

\( \alpha \) : Angle between leg of transverse reinforcement and core side crossed by the same leg
1. INTRODUCTION

1.1 Preamble

High Strength Concrete (HSC) has been increasingly used in construction industries, especially for the construction of lower storey columns of high-rise buildings and bridge piers, due to its beneficial engineering properties including strength and durability. However, low ductility and brittle failure offset the advantages of HSC columns (Foster 2001). It was found that columns constructed with HSC require a larger amount of lateral reinforcement compared to the columns constructed with Normal Strength Concrete (NSC) to attain similar ductility (Cusson and Paultre 1995, Razvi and Saatcioglu 1999). However, a large amount of lateral reinforcements may result in congestion of reinforcement causing problems of concrete placement and compaction (Cusson and Paultre 1995, Razvi and Saatcioglu 1999, Paultre et al. 2010).

Research studies have shown that the inclusion of steel fibres into HSC can improve the various mechanical properties of HSC including tensile strength, flexural strength, impact resistance and crack control. These improvements in the properties of HSC result from the ability of steel fibres to arrest cracks and redistribute the cracks under tensile stress. The main use of Steel Fibre Reinforced Concrete (SFRC) is in non-structural elements such as pools, slabs on grade and ground slope stabilization. Its use in structural elements (beams and columns), while permitted in some codes, is still the subject of research. Although, research studies on the behaviour of SFRC columns under concentric axial load are available, only limited research studies exist on the
behaviour of SFRC columns under eccentric axial load (Hadi 2009, Foster and Attard 2001).

This study presents the experimental and analytical results of sixteen circular HSC and SFR-HSC specimens tested under concentric axial load, eccentric axial load and four-point bending. From the experimental results, it was observed that the axial load and bending moment of SFR-HSC specimens were higher than those of HSC specimens. The analytical results of the specimens were in good agreement with the experimental results.

The brittle behaviour of concrete can be reduced by the inclusion of steel fibres (Zollo 1997, Bentur and Mindess 2006, Holschemacher et al. 2010). Hence, it was observed that the strength of concrete such as tensile strength, shear strength, and impact resistance improve with the inclusion of steel fibres (Batson 1976, Banthia and Mindess 1996, Gao et al. 1997, Bentur and Mindess 2006, Altun et al. 2007, Aydin 2013). In addition, the compressive strength and tensile strength of SFRC are two mechanical properties in the design of SFRC. The tensile strength of SFRC can be determined by different tests, for example, direct tensile test, flexural and splitting tensile tests. A number of research studies have studied the strength behaviour of SFRC (Wafa and Ashour 1992, Khaloo and Kim 1996, Song and Hwang 2004, Bentur and Mindess 2006, Thomas and Ramaswamy 2007, Yazici et al. 2007, Ramadoss and Nagamani 2008, Al-Hassani et al. 2014, Balanji et al. 2016). In general, they found that the splitting tensile strength, shear strength, and flexural strength of SFRC increase with the increasing volume content and aspect ratio (length/diameter) of steel fibres.
From the available research studies, it has been observed that quite a limited number of research studies are available to predict the axial compressive strength of the SFRC columns. Thus, it is essential to develop a new axial compressive strength model that accurately covers a wide range of the experimental results of the SFRC columns. In this study, to predict the axial compressive strength of the SFRC square columns, the confinement pressure due to lateral reinforcement and steel fibres are considered.

In this study, based on the available experimental investigation results, new compressive strength and splitting tensile strength models of SFRC were developed using Artificial Neural Network (ANN) analysis. In addition, a new compressive strength model of Steel Fibre Reinforced High Strength Concrete (SFR-HSC) columns were developed using ANN analysis.

1.2 Overview

In recent years, the use of HSC has steadily increased in structural applications, especially for the construction of lower storey columns of high-rise buildings, bridges, and foundation piles. The HSC exhibits higher durability and higher compressive strength than NSC. However, the advantages of using HSC in columns are offset by early cover spalling, reduction in ductility and brittle failure (Cusson and Paultre 1995, Foster and Attard 2001, Sharma et al. 2007a, Paultre et al. 2010). One of the techniques to increase the ductility and overcome the brittleness of concrete columns is the use of closely spaced lateral steel reinforcements (ties or helices) (Samaan et al. 1998, Razvi and Saatcioglu 1999). However, the lateral confinement provided by the helices or ties to the HSC columns is less effective compared to the lateral confinement provided by
the helices or ties to the NSC columns (Foster 2001). Therefore, significantly more lateral steel reinforcements are required for columns constructed with HSC to achieve similar strength and ductility enhancements. However, a high amount of lateral steel reinforcements may lead to early cover spalling due to the formation of the weak plane between the unconfined cover and confined core (Djumbong et al. 2008). This causes a temporary reduction in the load carrying capacity of the HSC columns.

Another technique to delay spalling of the concrete cover, increase ductility, and reduce the brittleness is the addition of macro steel fibres in HSC columns (Hadi 2007, Sharma et al. 2007b, Hadi 2009, Prisco et al. 2009, Paultre et al. 2010, Ou et al. 2011, Yoo et al. 2014). Macro steel fibre is defined as a fibre with a length higher than the maximum aggregate size and a diameter larger than that of cement grains (>50 µm) (Lofgren 2005). It was reported that the combination of macro steel fibres and lateral steel reinforcement can reduce the need for the comparatively high amount of lateral steel reinforcement required by design codes including ACI 318-05 (ACI 2005) and CSA23.3-04 (Canadian Standards 2004) for HSC columns (Paultre et al. 2010). This essentially means that the addition of macro steel fibres provides indirect confinement to the concrete core by delaying the spalling of concrete cover. Hence, enhancements in strength and ductility can be obtained for HSC columns. However, the major problem with the inclusion macro steel fibres is the reduction in the workability of the fresh concrete due to the high aspect ratio (length/diameter) and high volume content of the fibres. This limits the use of macro steel fibres in concrete to a maximum volume content of 2% (Huang et al. 2015). The limitation in the maximum amount of fibres in concrete causes limited improvements in the strength, modulus of elasticity, and strain
corresponding to the peak stress of the concrete (Bedirhanoglu et al. 2013, Huang et al. 2015).

Micro steel fibre, which is defined as a fibre with length less than the maximum aggregate size and the same diameter as the cement grain (≤ 50 μm) (Lofgren 2005), has been used in the composite concrete mix (Parikh et al. 2013). It was observed that the inclusion of 3% by volume of micro steel fibres into concrete lead to improvement in compressive strength, flexural strength and modulus of elasticity of the concrete (Yoo et al. 2014). It was also reported that the workability of the fresh concrete was not affected, as the aspect ratio of micro steel fibres was lower (Yoo et al. 2014). However, the behaviour of micro steel fibres reinforced HSC has not been adequately studied.

The failure in concrete is a multi-scale process (micro crack and macro crack). Hence, an optimal performance of concrete might not be achieved when one type of fibres (either macro or micro) is added into concrete (Huang et al. 2015). Thus, hybrid fibre, which is a combination of two or more types of fibres, was used into cementitious composites to optimize the properties of concrete material (Qian and Stroeven 2000, Kim et al. 2011, Akcay and Tasdemir 2012, Rambo et al. 2014, Chi et al. 2014, Huang et al. 2015), as well as to improve the performance of RC members (Ding et al. 2010, Issa et al. 2011, Ganesan et al. 2014). In general, the inclusion of hybrid fibres improved the shear strength and ductility of the beams and columns. The combination of hybrid fibres and lateral steel reinforcement may reduce the need for the high amount of lateral steel reinforcement in HSC columns.
The stress distribution in column cross-section under concentric load is uniform, and thus the confining stresses are equal. However, the stress distribution in the column cross-section under eccentric load is non-uniform, and hence the confining stresses are not equal (Foster and Attard 2001). Studies on the behaviour of macro steel fibres reinforced HSC column under eccentric load are limited. Foster and Attard (2001) found that the strength and ductility of the conventionally reinforced HSC column under eccentric load improve with the inclusion of 2% by volume of macro steel fibres. However, there is no investigation in the literature on the behaviour of micro and hybrid steel fibres reinforced HSC column under eccentric axial compression.

Previous research studies proposed compressive strength, splitting tensile strength and flexural strength models to investigate the influence of the types, volume content, and aspect ratio (length/diameter) of steel fibres on the strength behaviour of steel fibre reinforced concrete. In addition, the available strength models are based on regression analyses of the limited experimental investigation results. The available strength models of SFRC proposed based on a limited range of concrete compressive strength, volume content and aspect ratio of steel fibres.

From the literature review, it was observed that quite a limited number of research studied are available on the compressive strength models of SFRC columns (Paultre et al. 2010, Balanji et al. 2015). The available compressive strength models of SFRC columns proposed as functions of the compressive strength of unconfined concrete, confinement pressure provided by lateral reinforcement and steel fibres.
In this study, strength models of SFRC are proposed based on experimental investigation databases using ANN analysis. The proposed strength models cover a wide range of experimental investigation results. In addition, this study aims to determine the influence of different types of steel fibres (micro, macro and hybrid steel fibres) on the behaviour of HSC specimens under concentric axial load, eccentric axial load and four-point bending. 16 circular specimens of 200 mm diameter and 800 mm height were cast and tested to investigate the influence of the type, volume content and aspect ratio (length to diameter ratio) of steel fibres on the strength and ductility of HSC columns.

1.3 Objectives

The objectives of this research study can be briefly presented in the following points:

1. To develop a compressive strength model and a splitting tensile strength model of SFRC using ANN analysis. Also, to develop compressive strength models SFR-HSC square columns using ANN analysis. The proposed strength models cover a wide range of experimental investigation results.

2. To investigate the influence of volume content and aspect ratio of steel fibres (micro, macro and hybrid steel fibres) on the behaviour of HSC. Also, to find the optimum amount of the fibres.

3. To investigate the axial and bending behaviours of circular HSC and SFR-HSC specimens under concentric axial load, eccentric axial load and four-point bending.

4. To construct the analytical axial stress-strain curves of HSC and SFR-HSC.
5. To investigate the analytical behaviour of HSC and SFR-HSC circular specimens.

1.4 Outline of the Thesis

This study consists of nine chapters. The outline of these chapters is briefly presented below:

Chapter 1 presents an introduction of the effect of the inclusion of steel fibres into concrete columns, and the objectives of this research study.

In Chapter 2, a review of the available experimental investigation of concrete and SFRC, a review of the available compressive strength, splitting tensile strength and flexural strength models of SFRC, and a review of the available stress-strain models of concrete and SFRC are presented and discussed. A review of hybrid fibre reinforced concrete is also presented and discussed.

An overview of fibre reinforced concrete columns including the use of fibres in NSC and HSC column and the mechanism of confinement pressure provided by helix reinforcement and steel fibres are presented and discussed in Chapter 3. Also a review of stress-strain models of confined HSC and SFR-HSC columns are presented and discussed.

In Chapter 4, a database comprising 102 cylinders SFRC is compiled and analysed to propose a compressive strength and a splitting tensile strength models using ANN analysis. In addition, a database comprising 71 SFR-NSC and SFR-HSC square
columns is compiled and analysed to propose compressive strength models using ANN analysis.

Chapter 5 presents details of a pilot study regarding 40 cylindrical HSC and SFR-HSC.

In Chapter 6, details of the main experimental program regarding sixteen circular HSC and SFR-HSC columns are presented. The preliminarily testing of the materials used in the casting of the specimens are presented. The fabrication and testing of the specimens including, casting and curing of the specimens, instrumentation, and testing procedures are reported.

The experimental results of the preliminary testing of the materials and the main experimental results of the HSC and SFR-HSC specimens tested under concentric axial load, eccentric axial load and four-point bending are presented and discussed in Chapter 7.

In Chapter 8, the analytical axial load-bending moment interaction diagrams of HSC and SFR-HSC specimens using equivalent rectangular stress block method and the layer-by-layer integration method are presented. Also, the analytical bending moment-curvature curves of the SFR-HSC specimens tested under 25 mm and 50 mm eccentric axial loads are also presented.

In Chapter 9, the conclusions of this research study are presented. In addition, recommendations for future studies are listed.
2: FIBRE REINFORCED CONCRETE

2.1 General

Concrete is a brittle material, and it is prone to cracking under relatively small tensile stresses. The brittle and low tensile behaviours of the concrete can be improved with the inclusion of discontinuous discrete fibres. Type, volume content and physical properties of the fibres (length and diameter of the fibres) play an important role in characterising the mechanical properties such as tensile and flexural strength and toughness of the concrete. The mechanical properties of the concrete can be improved to a limited extent with the inclusion one type of steel fibres. However, it was found that the addition two or three types of fibres (hybrid fibres) into concrete can be more pronounced on the mechanical properties of the concrete.

This chapter presents a brief introduction of the compressive stress-strain behaviour of the plain concrete and fibre reinforced concrete as well as mechanical properties and tensile stress-strain of the fibre reinforced concrete. In addition, an introduction of the mechanical properties of the hybrid fibre reinforced concrete is also presented.

2.2 Review of Plain Concrete

The plain concrete is a composite material. It is made by mixing fine aggregate and coarse aggregate with Portland cement and water. The compressive strength of the concrete is quite higher than its tensile strength. The tensile strength of the concrete was ignored in the analysis of the concrete cross-section. Therefore, the classification of the concrete is based on their compressive strength. AS1379-15 (Australian Standard 2015) has defined normal-class concrete as a concrete having compressive strength grade up to
50 MPa and special-class concrete as a concrete having compressive strength grade greater than 50 MPa. ACI 363R-10 (ACI 2010) has stated that “40 MPa is dividing line between normal-strength and high strength concrete”. Thus, in this study, the compressive strength of concrete greater than 50 MPa was considered as High Strength Concrete (HSC).

The compressive stress-strain behaviour of the concrete depends on various parameters such as water-cement ratio, aggregate properties and loading rate. Hence, numerous studies have developed regression equations to model the compressive stress-strain behaviour of the concrete (Popovics 1973, Wang et al. 1978, Carreira and Chu 1985, Hsu and Hsu 1994a, Wee et al. 1996, Taerwe and Gysel 1996 and Lu and Zhao 2010) as shown in Tables 2.1. The compressive stress-strain behaviour of concrete comprises an ascending branch and a descending branch. The ascending branch starts at zero stress and finishes at the peak stress, and it is followed by the descending branch until the concrete crushes (ultimate stress). The initial tangent modulus, the peak stress and strain corresponding to the peak stress are the key parameters that are normally used to characterise the ascending and descending branches of the stress-strain curve of concrete. From the literature review, it was found that the compressive stress-strain models of concrete can be classified into two types. The first type (Type-1) consists of one model, which predicts the compressive stress-strain response for both ascending and descending branches from the point of origin to the strain corresponding to ultimate stress \((0 \leq \varepsilon_c \leq \varepsilon'_u)\). The second type (Type-2) consists of two models, the first model predicts the ascending branch from point of origin up to the limiting strain \((0 \leq \varepsilon_c \leq \varepsilon'_l)\), while the second model predicts the behaviour of the descending branch from limiting strain up to the strain corresponding to the ultimate stress \((\varepsilon'_l \leq \varepsilon_c \leq \varepsilon'_u)\).
Also, it was noted that the majority of the compressive stress-strain models are based on the stress-strain model that proposed by Popvics (1973). That models are modified the key parameters such as peak stress, strain corresponding to the peak stress and initial tangent modulus to present different concrete compressive stress-strain relationship.
Table 2.1. Regression-based compressive stress-strain models of unconfined concrete.

<table>
<thead>
<tr>
<th>References</th>
<th>Compressive stress-strain models</th>
<th>Key parameters</th>
<th>Compressive strength of concrete</th>
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<tbody>
<tr>
<td><strong>Type-1 model</strong></td>
<td></td>
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<tr>
<td>Popovics (1973)</td>
<td>( \frac{f_c}{f'_c} = \frac{\beta \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)}{\beta - 1 + \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)^\beta} )</td>
<td>( \beta = 0.058 f'_c + 1 )</td>
<td>Up to 51 MPa</td>
</tr>
<tr>
<td>Wang et al. (1978)</td>
<td>( \frac{f_c}{f'_c} = \frac{\beta \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)}{1 + \beta \left( \frac{\varepsilon_c}{\varepsilon'_c} \right) + \gamma \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)^2} )</td>
<td>A, B, C and D are constants which can be estimated by considering the condition ( f_c f'_c = 0.45 ) for ( \frac{\varepsilon_c}{\varepsilon'<em>c} = 0.45 ) ( \frac{E</em>{it}}{E'_c} ), ( \frac{f_c}{f'_c} = 1 ) for ( \frac{\varepsilon_c}{\varepsilon'_c} = 1 ).</td>
<td>Up to 76 MPa</td>
</tr>
<tr>
<td>Carriera and Chu (1985)</td>
<td>( \frac{f_c}{f'_c} = \frac{\beta \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)}{\beta - 1 + \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)^\beta} )</td>
<td>( \beta = \frac{1}{1 - \left( \frac{f'_c}{f'<em>c} / \frac{E</em>{it}}{E'_c} \right)} )</td>
<td>(23–80) MPa</td>
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<td></td>
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<td>( E_{it} = f'_c \left( \frac{24.82}{f'_c} + 0.92 \right) )</td>
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<td></td>
<td></td>
<td>( \varepsilon'_c = (1.680 + 7.1 f'_c) \times 10^{-6} )</td>
<td></td>
</tr>
<tr>
<td>Wee et al. (1996)</td>
<td>For ( f'_c \leq 50 ) MPa ( f_c = \frac{\beta \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)}{\beta - 1 + \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)^\beta} )</td>
<td>( \beta = \frac{1}{1 - \left( \frac{f'_c}{f'<em>c} / \frac{E</em>{it}}{E'_c} \right)} )</td>
<td>(50–120) MPa</td>
</tr>
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<td></td>
<td>For 50 MPa \leq f'_c \leq 120 MPa ( f_c = \frac{k_1 \beta \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)}{k_1 \beta - 1 + \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)^{k_2 \beta}} )</td>
<td>( E_{it} = 10.200 \left( f'_c \right)^{1/3} )</td>
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<td></td>
<td>( k_1 = \left( \frac{50}{f'_c} \right)^3 ; k_2 = \left( \frac{50}{f'_c} \right)^{1.3} )</td>
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Table 2.1. Continued

<table>
<thead>
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<th>References</th>
<th>Compressive stress-strain models</th>
<th>Key parameters</th>
<th>Compressive strength of concrete</th>
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<tr>
<td></td>
<td>Type-2 model</td>
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<tr>
<td>Hsu and Hsu (1994a)</td>
<td></td>
<td>$\beta = \left( \frac{f'_c}{65.23} \right)^3 + 2.59$</td>
<td>$\geq 69$ MPa</td>
</tr>
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<td></td>
<td>For $0 \leq \varepsilon_c \leq \varepsilon'_l$</td>
<td>$E_{it} = 0.0736 \times 10^{1.51} (f'_c)^{0.3}$</td>
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<td></td>
<td>$f_c = \frac{n \beta \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)^n}{n - 1 + \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)^n}$</td>
<td>For $0 \leq \varepsilon_c \leq \varepsilon'_l$; $n=1$</td>
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<td></td>
<td>For $\varepsilon'_l \leq \varepsilon_c \leq \varepsilon'_u$</td>
<td>For $\varepsilon'_l \leq \varepsilon_c \leq \varepsilon'_u$; $n=1$ if $f'_c &lt; 62$ MPa</td>
<td>Up to 90 MPa</td>
</tr>
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<td></td>
<td>$f_c = 0.3 f'_c \varepsilon^{-0.8} (\frac{\varepsilon_c}{\varepsilon'_c} \varepsilon'_c)^{0.5}$</td>
<td>n=2 if 62 MPa &lt; $f'_c &lt; 76$ MPa</td>
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<td></td>
<td>n=3 if 76 MPa &lt; $f'_c &lt; 90$ MPa</td>
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<td></td>
<td>n=5 if $f'_c \geq 90$ MPa</td>
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<td></td>
<td>$\varepsilon'_l$ is the strain at 0.3 $f'_c$ in the descending branch of the stress-strain curve.</td>
<td></td>
</tr>
<tr>
<td>Taerwe and Gysel (1996)</td>
<td>For $0 \leq \varepsilon_c \leq \varepsilon'_l$</td>
<td>$\eta_2 = \frac{\varepsilon'_l}{\varepsilon'<em>c} = \frac{1}{2} \left[ \frac{E</em>{it}}{2E_c} + 1 \right]$</td>
<td>(50–140) MPa</td>
</tr>
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<td>$f_c = \frac{E_{it}}{E_c} \left( \frac{\varepsilon_c}{\varepsilon'_c} - \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)^2 \right)$</td>
<td>$+ \frac{\left( \frac{E_{it}}{2E_c} + 1 \right)^2}{\eta_2 \left( \frac{E_{it}}{2E_c} - 2 \right) + 1}$</td>
<td></td>
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<tr>
<td></td>
<td>For $\varepsilon'_l \leq \varepsilon_c \leq \varepsilon'_u$</td>
<td>$\xi = 4 \eta_2 \left( \frac{E_{it}}{E_c} - 2 \right) + 2 \eta_2 - \frac{E_{it}}{2E_c}$</td>
<td></td>
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<tr>
<td></td>
<td>$f_c = \frac{E_{it}}{E_c} \left( \frac{\varepsilon_c}{\varepsilon'_c} - \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)^2 \right)$</td>
<td>$\eta_2 \left( \frac{E_{it}}{2E_c} - 2 \right) + 1$</td>
<td></td>
</tr>
<tr>
<td>Lu and Zhao (2010)</td>
<td>For $0 \leq \varepsilon_c \leq \varepsilon'_l$</td>
<td>$\frac{\varepsilon'_l}{\varepsilon_c} = \frac{1}{10} \left( \frac{1}{E_c} + \frac{4}{5} \right)$</td>
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<tr>
<td></td>
<td>$f_c = \frac{E_{it}}{E_c} \left( \frac{\varepsilon_c}{\varepsilon'_c} - \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)^2 \right)$</td>
<td>$+ \sqrt{\frac{1}{10} \left( \frac{1}{E_c} + \frac{4}{5} \right)^2 - \frac{4}{5}}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>For $\varepsilon'_l \leq \varepsilon_c \leq \varepsilon'_u$</td>
<td>$\lambda = \frac{1}{4}$</td>
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</table>

Compressive stress model

$\eta_2 = \frac{\varepsilon'_l}{\varepsilon'_c} = \frac{1}{2} \left[ \frac{E_{it}}{2E_c} + 1 \right]$

Compressive strength of concrete

$\xi = 4 \eta_2 \left( \frac{E_{it}}{E_c} - 2 \right) + 2 \eta_2 - \frac{E_{it}}{2E_c}$
2.3 Review of Fibre Reinforced Concrete

Fibre reinforced concrete is a special type of concrete. It is composed of fine aggregate, coarse aggregate, Portland cement, water and discontinuous discrete fibres. There are different types and geometries of the fibres that are commercially available such as straight, hooked end, crimped and deformed fibres. Lofgren (2005) classified the fibres used in cementitious composites into two groups. The first group is named as macro fibre. It includes the fibres which have a length larger than the maximum aggregate size, a cross-section diameter larger than that of cement grains (>50 µm) and an aspect ratio of less than 100. The second group is named as micro fibre. It includes the fibres which, has a length less than the maximum aggregate size and the same cross-section diameter as the cement grain. It was observed that the mechanical properties of the concrete such as tensile strength, flexural strength, impact strength, and toughness properties are significantly improved with the inclusion of fibres into concrete mixtures (Yazici et al. 2007). Fibres volume content and physical properties of fibres (length and diameter of the fibres) play an important role in characterising the mechanical properties of the concrete.

Wafa and Ashour (1992) studied the effect of hooked end steel fibre on the mechanical properties of the HSC. Hooked end steel fibre with a length of 60 mm and a diameter of 0.8 mm was used. Volume contents of the fibre of 0.5%, 1% and 1.5% were used. The authors found that the inclusion of 1.5% by volume of hooked end steel fibres into HSC led to slightly increasing in the compressive strength of the HSC by 4.6%. However, the flexural strength and splitting tensile strength significantly increased by 67% and
159.8%, respectively. It was also found that the workability of the steel fibre reinforced concrete did not reduce with the inclusion 1.5% of the fibres.

Kovler et al. (1992) found that the plastic shrinkage of fibre reinforced concrete is significantly decreased due to the inclusion of polypropylene fibres. Regarding the total shrinkage of fibre reinforced concrete, they found that the effect of polypropylene fibre reinforcement was insignificant up to a volumetric content of 0.2%. According to their findings, crack width can be reduced as much as 50% by increasing the volumetric content of polypropylene fibres.

Bayasi and Zeng (1993) examined the behaviour of Polypropylene Fibre Reinforced Concrete (PFRC). They used both short (12.7 mm) and long (19 mm) polypropylene fibres with volume contents of 0.1%, 0.3% and 0.5%. They found that by adding short fibres, the compressive strength was increased by 15% to 19% for the volume contents in the range of 0.1% to 0.3%. Moreover, the addition of long fibres has no influence on the compressive strength of the PFRC. They also found that the flexural strength of PFRC including long fibres was slightly more effective than short fibres for volume contents of 0.3% or less. Finally, the long fibres were more effective for volume contents of 0.3% or less, while the short fibres were more effective for 0.5% volume content.

Chen and Chung (1993) investigated the performance of Carbon Micro-Fibre (CMF) reinforced concrete. CMF with a length of 5 mm and volume content of 0.2% was used. The study concluded that compressive strength, flexural strength and toughness were increased by 22%, 85% and 205%, respectively. The study also concluded that
durability to freeze-thaw resistance was improved, and drying shrinkage and electrical resistivity decreased by 90% and 83%, respectively.

Khaloo and Kim (1996) investigated the influence of hooked end steel fibres on the mechanical properties of the Normal Strength Concreter (NSC), Medium Strength Concrete (MSC) and HSC. Hooked end steel fibres with a diameter of 0.55, a length of 32 mm and fibre volume contents of 0.5%, 1% and 1.5% were used. They observed that the maximum improvement in the compressive stress of NSC was achieved with the addition of 1.5% by volume of the fibres into the concrete. Whereas, the maximum enhancement in the compressive stress of the MSC and HSC was found with the inclusion of 1% by volume of the fibres into the concrete. It was found that the splitting tensile strength improved for the NSC, MSC and HSC. It was also found that the inclusion of the fibres into NSC, MSC and HSC provided a higher improvement in the flexural strength of the NSC compared to MSC and HSC.

Song and Hwang (2004) studied the effect of hooked end steel fibres on the mechanical properties of the HSC. Hooked end steel fibres with a diameter of 0.55, a length of 35 mm and fibre volume content of 0.5%, 1%, 1.5% and 2% were used. The authors found that the maximum compressive stress of the HSC was observed with the adding 1.5% of the fibres into HSC. However, the maximum compressive stress slightly reduced with the adding 2% of the fibres. They also found that the splitting tensile strength and flexural strength increased with an increasing of the volume content of the fibres. Maximum improvement in the splitting tensile strength and flexural strength was observed with the adding 2% of the fibres.
Yazici et al. (2007) investigated the effect of the aspect ratio and the volume content of hooked end steel fibres on the mechanical properties of the NSC. Three aspect ratios of 45, 65 and 80 were used. The volume content of 0.5%, 1% and 1.5% of the fibres were used. The authors found that steel fibre reinforced concrete with a fibre volume of 1.5%, 1% and 0.5% have the highest compressive stress for the aspect ratio of 45, 65 and 80, respectively. The authors also observed that the splitting tensile strength of steel fibre reinforced concrete increased with an increasing aspect ratio of the fibres. It was also observed that the flexural strength of steel fibre reinforced concrete significantly improved with an increasing aspect ratio and volume content of the fibres.

Thomas and Ramaswamy (2007) studied the effect of the addition of hooked end steel fibre on the mechanical properties of the NSC (35 MPa), MSC (65 MPa) and HSC (85). Hooked end steel fibre with a diameter of 0.55, a length of 30 and fibre volume content of 0.5%, 1% and 1.5% were used. It was found that the compressive strength, modulus of elasticity and Poisson’s ratio of the NSC, MSC and HSC slightly increased with the addition of the fibres (less than 10%). It was also found that the maximum increase in splitting tensile strength and flexural strength of the NSC, MSC and HSC due to the inclusion of the fibres was about 40%.

Ramadoss and Nagamani (2008) studied the influence of crimped steel fibres on the mechanical properties of the concrete. The concrete compressive strength ranged from 45 MPa to 85 MPa. Crimped steel fibres having a diameter of 0.45 mm and a length of 36 mm were used. The volume content of 0.5%, 1% and 1.5% of the fibres were used. It was observed that the addition of steel fibres by 1.5% into concrete results in 12.4% increasing in the compressive strength, an increase of 38% in the flexural strength and
an increase of 56% in the splitting tensile strength compared to concrete with no steel fibres.

Parikh and Dhyani (2013) investigated the influence of micro steel fibre on the compressive strength of concrete containing silica fume. Micro steel fibre with a diameter of 0.17 mm, a length of 6 mm and fibre volume content of 0.5%, 1%, 1.5% and 2% were used. It was found that no problem of balling of fibre occurred due to the lower aspect ratio of the fibre. It was also observed that the addition of micro steel fibre and silica fume resulted in an increasing in the compressive strength of the concrete by 30%.

Balanji et al. (2016) reported the results of an experimental investigation on the behaviour of HSC reinforced with different types of steel fibre (micro steel fibre, and macro steel fibre). A total of 40 cylindrical specimens of 100 mm x 200 mm were cast and tested for compressive strength and splitting tensile strength. Two types of steel fibre reinforced HSC specimens were prepared. The first type included 2%, 3%, and 4% by volume of micro steel fibres. The second type included 1%, 2% and 3% by volume of macro steel fibres. It was observed that the maximum improvement in the compressive stress of HSC was observed with the inclusion of 3% of micro steel fibres and 2% of macro steel fibres. Also, the higher improvement in splitting tensile strength was observed with the inclusion of 4% and 3% by volume of micro steel fibre and macro steel fibre, respectively.
2.4 Regression-Based Strength of Steel Fibre Reinforced Concrete Models

The previous research studies proposed strength models of compressive strength, and tensile strength to determine the effectiveness of parameters, such as concrete compressive strength, volume content, and aspect ratio of steel fibres on the strength behaviour of Steel Fibre Reinforced Concrete (SFRC). In general, it was found that the improvements in the strength of SFRC can be a function of the different parameters, such as concrete compressive strength, volume content, aspect ratio, and type of steel fibres. In addition, the available strength models based on regression analyses of limited experimental investigation results. The available strength models have proposed based on a limited range of concrete compressive strength, volume content and aspect ratio of steel fibres to develop strength models of SFRC. Table 2.2 presents the available strength models of SFRC. It can be seen from Table 2.2 that the strength models proposed by Wafa and Ashour (1992), Khaloo and Kim (1996), Song and Hwang (2004) and Al-Hassani et al. (2014) considered the effect of volume content of steel fibres. They did not consider the effect of aspect ratio of steel fibres on the strength of SFRC. Yazici et al. (2007) examined the effect of volume content and aspect ratio of steel fibres as separate two parameters. They found that each parameter of steel fibres has different contribution effect on the strength of SFRC. The strength models that proposed by Thomas and Ramaswamy (2007) and Ramadoss and Nagamani (2008) have considered the effect of the interaction of volume content and aspect ratio of steel fibres, i.e., fibre reinforced index on the strength of SFRC. In addition, the available strength models were validated for a limited range of experimental investigation results and specific type of steel fibres such as hooked-end and cramped steel fibres. It was found that the available strength models were in reasonable agreement with their experimental investigation results. However, the errors estimating in the available
strength models for a limited range of experimental investigation results still exist. It is indicated that there are different arguments about the use of parameters such as volume content and aspect ratio of steel fibres to determine the strength of SFRC.

Artificial Neural Network (ANN) analysis is used in Chapter 4 to propose compressive strength and splitting tensile strength models that cover wide range of the experimental investigation results. Further details about ANN analysis are presented and discussed in Chapter 4.
<table>
<thead>
<tr>
<th>References</th>
<th>Compressive strength</th>
<th>Splitting tensile strength</th>
<th>Flexural strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wafa and Ashour (1992)</td>
<td>( f'_{cf} = f'_c + 3.53 \nu_f )</td>
<td>( f'_{sp} = 0.58 \sqrt{f'_c} )</td>
<td>( f'_{tf} = 1.03 \sqrt{f'_c} )</td>
</tr>
<tr>
<td>Khaloo and Kim (1996)</td>
<td>( f'_{cf} = f'_c (1 + 0.568 \nu_f - 0.303 \nu_f^2) )</td>
<td>( f'_{spf} = (0.67 + 0.1256 \nu_f - 0.0116 \nu_f^2) \sqrt{f'_c} )</td>
<td>( f'_{rf} = (0.68 + 0.206 \nu_f - 0.085 \nu_f^2) \sqrt{f'_c} )</td>
</tr>
<tr>
<td>Song and Hwang (2004)</td>
<td>( f'_{cf} = 85 + 15.12 \nu_f - 4.7 \nu_f^2 )</td>
<td>( f'_{spf} = 5.8 + 3.01 \nu_f - 0.02 \nu_f^2 )</td>
<td>( f'_{rf} = 6.4 + 3.43 \nu_f - 0.32 \nu_f^2 )</td>
</tr>
<tr>
<td>Yazici et al. (2007)</td>
<td>( f'_{cf} = 50.4869 + 0.0434 \times \frac{l_f}{d_f} ) + 1.9667 \times \nu_f )</td>
<td>( f'_{spf} = 2.2121 + 0.0077 \times \frac{l_f}{d_f} + 1.4233 \times \nu_f )</td>
<td>( f'_{rf} = 0.8261 + 0.0638 \times \frac{l_f}{d_f} + 3 \times \nu_f )</td>
</tr>
<tr>
<td>Thomas and Ramaswamy (2007)</td>
<td>( f'_{cf} = 0.84 f'_c + 0.046 f'_c R.I. + 1.02 R.I. )</td>
<td>( f'_{spf} = 0.63 (f'_c)^{0.5} + 0.288 (f'_c)^{0.5} R.I. + 0.052 R.I. )</td>
<td>( f'_{rf} = 0.97 (f'_c)^{0.5} + 0.295 (f'_c)^{0.5} R.I. + 1.117 R.I. )</td>
</tr>
<tr>
<td>Ramadoss and Nagamani (2008)</td>
<td>( f'_{cf} = f'_c + 1.498 R.I. )</td>
<td>( f'_{spf} = 0.57 \sqrt{f'_c} \text{ For } 30 \text{ MPa} &lt; f'_c &lt; 75 \text{ MPa} )</td>
<td>( f'_{rf} = 0.79 \sqrt{f'_c} \text{ For } 30 \text{ MPa} &lt; f'_c &lt; 75 \text{ MPa} )</td>
</tr>
<tr>
<td>Al-Hassani et al. (2014)</td>
<td>None</td>
<td>( f'_{spf} = 0.024 f'_c + 2.614 \nu_f )</td>
<td>None</td>
</tr>
</tbody>
</table>

\( \frac{l_f}{d_f} \) refers to the aspect ratio of steel fibre, where \( l_f \) refers to the length of steel fibre, and \( d_f \) refers to the diameter of steel fibre.
2.5 Compressive Stress-Strain Behaviour for Fibre Reinforced Concrete

The compressive stress-strain models of fibre reinforced concrete were modified from compressive stress-strain models of concrete. It is necessary to modify the key parameters (peak stress, strain corresponding to the peak stress and initial tangent modulus) in the stress-strain model of concrete in order to consider the influence of fibres on the stress-strain curve. Table 2.3 presents the available compressive stress-strain models of SFRC. The available compressive stress-strain models of SFRC are presented and discussed in the following sections.

2.5.1 Model Proposed by Ezeldin and Balaguru (1992)

Ezeldin and Balaguru (1992) modified the compressive stress-strain model proposed by Carreira and Chu (1985) to determine the influence of fibre on the compressive stress-strain behaviour of SFRC. An analytical expression of the peak stress, strain corresponding to the peak stress, initial tangent modulus and the toughness of the concrete were proposed by taking into account the effect of the steel fibre reinforced index (R.I.) (interaction between volume content and aspect ratio of steel fibres (length/diameter). Ezeldin and Balaguru (1992) used hooked end steel fibre in their study. The R.I. ranged from 0.23 to 0.77. The compressive strength of the control specimen was 36 MPa. Ezeldin and Balaguru (1992) also proposed different materials parameters, β, for the hooked end and straight steel fibres as shown in Table 2.3. It was obtained that the proposed analytical expressions provided a good agreement between predicted and experimental results.
2.5.2 Model Proposed by Hsu and Hsu (1994b)

Based on the compressive stress-strain model of HSC proposed by Hsu and Hsu (1994a), Hsu and Hsu (1994b) modified a compressive stress-strain model for steel fibre reinforced HSC. Hsu and Hsu (1994b) proposed new material parameters $\beta$ and $n$. Factor $\beta$ depends on the shape of the stress-strain diagram, and $n$ depends on the strength of material. The compressive strength ranged from 66 MPa to 88 MPa. The volume contents of the steel fibre in the concrete mixes were 0%, 0.5%, 0.75% and 1%. These various parameters were studied, and their relationships were experimentally determined. The analytical expressions of the material parameters, strain corresponding to the peak stress and initial tangent modulus were used to define the complete stress-strain relationship for the steel fibre reinforced HSC subjected to uniaxial compression are presented in Table 2.3.

2.5.3 Model Proposed by Mansur et al. (1999)

Mansur et al. (1999) modified compressive stress-strain model proposed by Carreira and Chu (1985) to develop a compressive stress-strain model for the steel fibre reinforced HSC. The compressive strength of the control specimens was 70 MPa. Hooked end steel fibres with $R.I.$ ranged from 0.3 to 0.9 were used in their study. Mansur et al. (1999) introduced new factors ($k_1$ and $k_2$) in the stress-strain model which were dependent on the effect of different parameters such as compressive strength of concrete, cast direction (horizontal cast prisms and vertical cast prisms) of the concrete and volume content of the fibre. Based on different cast directions, the authors proposed an initial tangent modulus and a strain corresponding to the peak stress as shown in Table 2.3. The proposed analytical models were validated by the experimental test data.
of cylinders and prisms specimens. It was observed that the addition of the fibres enhances the strength and strain at the peak stress.

2.5.4 Model Proposed by Nataraja et al. (1999)

Nataraja et al. (1999) modified compressive stress-strain model proposed by Carreira and Chu (1985) to quantify the effect of the fibre on the compressive strength, the strain corresponding to the peak stress and the toughness of concrete in terms of fibre reinforced parameters. Nataraja et al. (1999) proposed analytical expression of the peak stress, strain corresponding to the peak stress, initial tangent modulus and toughness of the concrete as a function of the fibre reinforced index, which is the interaction between volume content and aspect ratio of the fibres, as shown in Table 2.3. The proposed analytical expression was based on the experimental investigations, which were used to develop the complete stress-strain curve of SFRC for compressive strength ranged from 29 MPa to 43 MPa. Crimped steel fibres with the R.I. ranged from 0.28 to 0.82 were used in their study. It was found that the analytical expression provided a good agreement between predicted and experimental results.

2.5.5 Model Proposed by Bhargava et al. (2006)

Based on the experimental test data, Bhargava et al. (2006) modified the compressive stress-strain model proposed by Carreira and Chu (1985) to develop a complete stress-strain relationship for steel fibre reinforced HSC. Bhargava et al. (2006) introduced two factors (k₁, k₂) in the stress-strain model. These factors are used to develop the best fit for the ascending branch (0 \( \leq \varepsilon_c \leq \varepsilon'_c \)) of k₁=k₂=1, and for the descending branch (\( \varepsilon'_c \leq \varepsilon_c \leq \varepsilon'_u \)) by calculating k₁ and k₂ from the equations presented in Table 2.3.
Bhargava et al. (2006) also proposed peak stress, strain corresponding to the peak stress, initial tangent modulus and toughness of the concrete by taking into account the effect of the fibre reinforced index for the short and long steel fibres as shown in Table 2.3. The experimental program consisted of testing 100 mm × 200 mm concrete cylinders. The experimental variables in that study were concrete strength of 58.03 MPa and 76.80 MPa, the R.I. ranged from 0.1 to 0.8 of flat crimped steel fibre. A good agreement has been achieved between the predicted and experimental results.

2.5.6 Model Proposed by Oliveira et al. (2010)

Oliveira et al. (2010) modified the compressive stress-strain model proposed by Carreira and Chu (1985) to determine a compressive stress-strain behaviour of SFRC. Based on nonlinear regression analyses, a new material parameter (β) and strain corresponding to the peak stress were proposed as a function of variation of compressive strength and volume content of steel fibre as shown in Table 2.3. In their study, the compressive stress-strain curve for steel fibre reinforced concrete was derived from the compressive strength of 40 MPa and 60 MPa at the age of 28 days. Hooked end steel fibres with volume content of 1% and 2% were used to reinforced the concrete. The aspect ratio was 56.

2.5.7 Model Proposed by Ou et al. (2011)

Ou et al. (2011) modified the compressive stress-strain model of concrete proposed by Carreira and Chu (1985) to calculate a stress-strain model of SFRC. Ou et al. (2011) proposed analytical expression of the peak stress, strain corresponding to the peak stress and material parameter (β) of the SFRC as a function of the R.I. as shown in Table 2.3.
The proposed analytical expression was based on the experimental investigations, which were used to develop the complete compressive stress-strain curve of SFRC. The compressive strength of the control specimen was 40 MPa. Hooked end steel fibres with the R.I. ranged from 0.4 to 1.7 were used in their study. It was found that the analytical expression provided a good agreement between predicted and experimental results. It was also found that the analytical models proposed of the compressive stress-strain behaviour of SFRC in compression are applicable to the R.I. up to 1.7 of steel fibres.
Table 2.3. Key parameters to defined compressive stress-strain models of the SFRC.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Compressive stress-strain models</th>
<th>Key parameters to defined stress-strain models</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ezeldin and Balaguru (1992)</td>
<td>$f_c = \frac{\beta (\frac{\varepsilon_c}{\varepsilon_c^\prime})}{\beta - 1 + (\frac{\varepsilon_c}{\varepsilon_c^\prime})}$ For hooked ends steel fibre $\beta = 1.093 + 0.7132(R.I.)^{-0.926}$ $f_c^\prime = f_c^\prime + 3.51(R.I.)$ For straight steel fibre $\beta = 1.093 + 7.4818(R.I.)^{-1.387}$ $R.I. = n \times \left(\frac{l_f}{d_f}\right)$</td>
<td></td>
</tr>
<tr>
<td>Hsu and Hsu (1994b)</td>
<td>$f_c^\prime = \frac{n \beta (\frac{\varepsilon_c}{\varepsilon_c^\prime})}{\beta - 1 + (\frac{\varepsilon_c}{\varepsilon_c^\prime})}$ For $0 \leq \varepsilon_c \leq \varepsilon'_l$ $\beta = \left[\frac{\varepsilon_c^\prime}{\varepsilon_c}\right]^3 + 8.501$ $\varepsilon'_lt = a_2 f'_c + C_2$ $\frac{n}{n} \leq \varepsilon \leq \varepsilon'_l$ $n=1$, if $f_c^\prime &lt; \varepsilon'_l$ $n=1.5$, if $79 \leq f_c^\prime &lt; 83$ $n=2$, if $83 \leq f_c^\prime$ $\beta = -0.26 \nu_f + 2.742$ $C = a_3 f'_c + C_1$ $\nu_f = 0.00142$ $0.00183$ $0.75$ $0.000118$ $0.00217$ $1.0$ $0.000178$ $0.00164$ $\frac{n}{n}$ $0.5$ $0.75$ $1.0$ $n=1$ $n=1.5$, if $79 \leq f_c^\prime &lt; 83$ $n=2$, if $83 \leq f_c^\prime$ $</td>
<td>\varepsilon_c</td>
</tr>
<tr>
<td>Nataraja et al. (1999)</td>
<td>$f_c = \frac{\beta (\frac{\varepsilon_c}{\varepsilon_c^\prime})}{\beta - 1 + (\frac{\varepsilon_c}{\varepsilon_c^\prime})}$ $\beta = 0.5811 + 1.93 (R.I.)^{-0.7406}$ $f_c^\prime = f_c^\prime + 2.1604 (R.I.)$ $\varepsilon'_lt = \varepsilon'<em>c + 0.0006 (R.I.)$ $E</em>{lt} = 1930 (R.I.)^{-0.7406}$</td>
<td></td>
</tr>
<tr>
<td>Mansur et al. (1999)</td>
<td>$f_c = \frac{\beta (\frac{\varepsilon_c}{\varepsilon_c^\prime})}{\beta - 1 + (\frac{\varepsilon_c}{\varepsilon_c^\prime})}$ $\beta = \left[\frac{\varepsilon_c^\prime}{\varepsilon_c}\right]^3$ Cast direction Horizontally cast prisms $k_1 = A \left(\frac{40}{f_{ct}^\prime}\right)^3, k_2 = B \left(\frac{40}{f_{ct}^\prime}\right)^{13}$ $A=1.79$ for Plain concrete. $A=0.96$ and $B=0.8$ for fibre concrete. Horizontally cast prisms $k_1 = A \left(\frac{40}{f_{ct}^\prime}\right)^{13} \left[1 + 2.5 (R.I.)^{2.5}\right]$ $k_2 = B \left(\frac{40}{f_{ct}^\prime}\right)^{13} \left[1 - 0.11 (R.I.)^{-1.1}\right]$ Vertically cast prisms $\varepsilon'<em>ct = 0.00048 (f</em>{ct}^\prime)^{0.35}$ $E_{lt} = 10.300 (f_{ct}^\prime)^{1.3}$ $\varepsilon'<em>ct = 0.00048 (f</em>{ct}^\prime)^{0.35}$ $E_{lt} = 10.300 (f_{ct}^\prime)^{1.3}$</td>
<td></td>
</tr>
</tbody>
</table>
Table 2.3. Continued

<table>
<thead>
<tr>
<th>Reference</th>
<th>Compressive stress-strain models</th>
<th>Key parameters to defined stress-strain models</th>
</tr>
</thead>
</table>
| Bhargava et al. (2006) | $f_c = \frac{k_1 \beta \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)}{k_1 \beta - 1 + \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)^\beta k_2}$ | $(0 \leq \varepsilon \leq \varepsilon'_c)$  
$\beta = \left[ \frac{\varepsilon_c}{\varepsilon'_c} \right]^3 + C$; $k_1 = k_2 = 1$; $A = 50.35 + 22.31 (R.I.)_{\text{short}} + 19.13 (R.I.)_{\text{long}}$; $C = 2.04 - 0.313 (R.I.)_{\text{short}} - 0.155 (R.I.)_{\text{long}}$  
$(\varepsilon'_c \leq \varepsilon \leq \varepsilon'_u)$  
$k_1 = A \left( \frac{\sigma}{\sigma_{\text{cf}}} \right)^{-3.79}, k_2 = B \left( \frac{\sigma}{\sigma_{\text{cf}}} \right)^{1.46}$; $D = 35.635 + 17.21 (R.I.)_{\text{short}} + 9.11 (R.I.)_{\text{long}}$;  
$G = 31.82 + 16.39 (R.I.)_{\text{short}} + 9.35 (R.I.)_{\text{long}}$  
$f'_{\text{cf}} = f'_c + 0.45 + 8.89 (R.I.)_{\text{short}} + 2.47 (R.I.)_{\text{long}}$  
$\varepsilon'_{\text{cf}} = \varepsilon'_c - 0.00026 + 0.0001214 (R.I.)_{\text{short}} + 0.00086 (R.I.)_{\text{long}}$  
$E_{\text{icf}} = E_i + 422 - 603 (R.I.)_{\text{short}} - 597 (R.I.)_{\text{long}}$ |
| Oliveira et al. (2010) | $f_c = \frac{\beta \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)}{\beta - 1 + \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)^\beta}$ | $\beta = (0.0536 - 0.578 \nu) f'_{\text{cf}}$  
$\varepsilon'_{\text{cf}} = (0.00048 + 0.01886 \nu) \ln f'_{\text{cf}}$ |
| Ou et al. (2011) | $f_c = \frac{\beta \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)}{\beta - 1 + \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)^\beta}$ | $\beta = 0.71 (R.I.)^2 - 2 (R.I.) + 3.05$  
$f'_{\text{cf}} = f'_c + 2.35 (R.I.)$  
$\varepsilon'_{\text{cf}} = \varepsilon'_c + 0.0007 (R.I.)$ |
2.6 Tensile Stress-Strain Behaviour for Fibre Reinforced Concrete

The plain concrete fails in a brittle manner under a tension load. The inclusion of fibres into plain concrete results in improving the tensile capacity. Lok and Xiao (1998) proposed tensile stress-strain behaviour of fibres reinforced concrete tested under direct tension load as shown in Figure 2.1.

![Tensile Stress-Strain Curve](image)

Figure 2.1. Proposed constitutive tensile stress-strain relationship (Lok and Xiao 1998).

The authors divided the tensile stress-strain curve of the fibre reinforced concrete into two distinct stages, i.e., pre-cracking stage and post-cracking stage. The ultimate tensile stress and corresponding tensile strength are the transitional cracking point of the fibre...
reinforced concrete. In the pre-cracking stage, the influence of the matrix cracking is ignored. It is assumed that the fibre perfectly bonds with the concrete and no slippage occurs at the fibre-matrix interface. The initial tangent modulus of concrete in tension and compression is assumed to be equal. The fibres are strained when the cracks occur. Lok and Xiao (1998) proposed stress-strain model to calculate the parabolic tensile curve in the pre-cracking stage and is expressed as follows:

\[
 f_t = f_{ct} \left[ 2 \left( \frac{\varepsilon_t}{\varepsilon_{ct}} \right) - \left( \frac{\varepsilon_t}{\varepsilon_{ct}} \right)^2 \right] \tag{2.1}
\]

where \( f_{ct} \) and \( \varepsilon_{ct} \) are the ultimate tensile stress and corresponding tensile strain, respectively, and are calculated as follows:

\[
 f_{ct} = f_m [1 - v_f] + \eta_0 \eta_\theta \tau_f v_f l_f \tag{2.2}
\]

\[
 \varepsilon_{ct} = 2 \left( \frac{f_{ct}}{E_{ct}} \right) \tag{2.3}
\]

where \( f_m \) is the tensile stress of the matrix, \( \eta_0 \) is fibre length factors, \( \eta_\theta \) is the fibre orientation factor, respectively, \( \tau_f \) is the bond shear stress of the steel fibres.

The post-cracking stage occurs after ultimate tensile stress and corresponding strain. In this stage, the fibres are progressively strained due to arresting and bridging the cracks. This leads to significantly improving the tensile capacity of the concrete. In order to
simplify the post-cracking stage behaviour, Lok and Xiao (1998) adopted the bilinear process in the proposed tensile model. The first process is Process I. This process starts from the points $f_{ct}$ and $\varepsilon_{ct}$ to the point $f_{rt}$ and $\varepsilon_{rt}$ as shown in Figure 2.1. Where $f_{rt}$ and $\varepsilon_{rt}$ is the residual tensile stress and corresponding strain, respectively. In Process I, the bond shear stress is gradually developed, while the fibres are being strained. The bond shear stress is fully developed at the end of the Process I. Lok and Xiao (1998) proposed the $f_{rt}$ and $\varepsilon_{rt}$ as follows:

$$f_{rt} = \eta_f \tau_f \frac{v_l f}{d_f} \quad (2.4)$$

$$\varepsilon_{rt} = \tau_f \left[ \frac{l_f}{d_f} \frac{1}{E_{sf}} \right] \quad (2.5)$$

The second process is Process II. This process starts from the point $f_{rt}$ and $\varepsilon_{rt}$ to the failure tensile strain ($\varepsilon_{ut}$). It was found that the nominal changes in value of the $\varepsilon_{ut}$ has insignificant effect on the post-cracking behaviour of fibre reinforced concrete (Lok and Xiao 1998).

### 2.7 Review of Hybrid Fibre Reinforced Concrete

The inclusion of two or three different types of fibre into concrete is an effective method for maximizing the strength enhancements through fibre reinforcement, which makes complementary contributions to performance in a concrete composites mixtures (Banthia and Sheng 1990, Betterman et al. 1995, Mobasher and Li 1996, Qian and Stroeven 2000, Bentur and Mindess 2006).
Lawler et al. (2003) found that a blend of micro fibre having a diameter less than 0.022 mm and a macro fibre having a diameter of 0.5 mm in a mortar mixture has a positive effect on reducing the crack growth at different stages of failure process. The authors also found that the combination of micro fibre and macro fibre (hybrid fibre) leads to improvements in the mechanical properties of concrete such as compressive strength and tensile strength which might not be achievable with the use of macro fibre alone in the mixture.

Banthia and Gupta (2004), and Banthia and Soleimani (2005) categorised the hybrid fibres into three groups. The first group is blending different positive constitutive responses of the fibres. The first is stiffer or stronger fibre which provides stiffer and reasonable first cracks strength and ultimate strength, and the second is flexible fibre which provides an improvement in the strain capacity in the post cracking zone, and also leads to improvement in the toughness of materials. The second group is blending different aspect ratios of the fibres. The short fibres bridge the micro-cracks, which delays the crack growth. However, the long fibres arrest the propagation of macro-cracks and hence improve the fracture toughness of the composites. The third group is considering the function of the fibres by blending micro synthetic fibres with steel fibres, or micro synthetic fibres with macro synthetic fibres. In this combination, one type of fibre improves fresh, and early properties of the fibre reinforced concrete, such as workability and control the effect of the plastic shrinkage cracks growth in the fibre reinforced concrete, and the second type of fibre improves the mechanical properties of the fibre reinforced concrete.
Hong and Choi (2012) studied the tensile behaviour of steel–carbon hybrid fibre reinforced cementitious composites. It was found that carbon fibres improved the toughness index in tensile behaviour before reaching the maximum crack opening, while steel fibres improved the tensile strength. An increase in the steel fibre content with a decrease in the carbon fibre content resulted in an increase in the tensile strength and decrease in the toughness index. Since the tensile behaviour at the large crack width is predominantly governed by the steel fibres, the fracture energy is mainly dependent on the steel fibre content, rather than the carbon fibre content.

Park et al. (2012) studied the influence of blending macro fibres and micro fibres on the tensile behaviour of ultra-high performance concrete. Four different types of macro steel fibre were used having volume content of 1%, while the volume content of micro steel fibres was varied from 0% to 1.5%. They observed that tensile properties were significantly improved. Moreover, the number of the micro-cracks was considerably reduced by increasing the volume content of micro steel fibres.

Yusof et al. (2011) studied the behaviour of Hybrid Steel Fibre Reinforced Concrete (HSFRC), by considering different aspect ratios and volume contents. It was proven that HSFRC having a volume content of 1.5%, consisting of 30% of long fibres and 70% of short fibres, produced the highest value of compressive strength. On the other hand, the highest improvements in flexural strength and split tensile strength were obtained by the inclusion of 1.5% of steel fibre, consisting of 70% of long fibre and 30% of short fibre.

Dawood and Ramli (2011) investigated the behaviour of high strength mortar, mortar reinforced with the steel fibres, and hybrid fibres consisting of steel fibres, palm fibres
and synthetic fibres (Bar-chip). It was found that the best hybridization of fibres was obtained in the quantities ranging from volume content of 1.5% to 1.75% of steel fibres with volume content of 0.25% to 0.5% of either palm or Bar-chip fibres. This hybridization resulted in the highest values of compressive strengths, approximately 45% and 140% increase in splitting tensile strength and flexural strength, respectively for the composites.

Liu et al. (2012) investigated the behaviour of hybrid fibre reinforced HSC. Steel fibre and polypropylene fibre with the volume content of 1.2% of steel and the volume content 0.1% of polypropylene fibres were used. This significantly reduces the brittleness of HSC and prevents spalling of the concrete cover. It was also concluded that the combination of steel and polypropylene fibres resulted in improvement in the compressive strength of HSC.

Chi et al. (2014) proposed equations to calculate the compressive strength and strain corresponding peak stress of Steel-Polypropylene Hybrid Fibre Reinforced Concrete (SPHFRC), by considering the variation in the volume contents and the aspect ratio of the fibres.

\[
f_{cf}' = f_c'(1 + 0.206 R.I.sf + 0.388 R.I.pf) \tag{2.6}
\]

\[
e_{cf}' = e_c'(1 + 0.705 R.I.sf + 0.364 R.I.pf) \tag{2.7}
\]

where \(f_{cf}'\) is the compressive strength of SPHFRC, \(f_c'\) is the compressive strength of the concrete without fibre reinforcement, \(R.I.sf\) and \(R.I.pf\) are the reinforced index of steel
and polypropylene fibres, respectively. The reinforced index of fibres is calculated as

\[ R.I.t = v_f \times \frac{l_f}{d_f} \]

where \( v_f \) is the volume content of fibres; and \( l_f/d_f \) is the aspect ratio of fibres. \( \varepsilon_{ct}^{\prime} \) is the peak strain of the steel fibre reinforced concrete at peak stress and \( \varepsilon_{c}^{\prime} \) is the peak strain of the concrete without fibre reinforcement at peak stress.

Balanji et al. (2016) investigated the compressive stress-strain behaviour of Hybrid Steel Fibre Reinforced High Strength Concrete (HSFR-HSC). Three different combinations of HSFR-HSC specimens and reference specimens without steel fibres were prepared. The first combination of HSFR-HSC included 1.5% Micro Steel (MS) fibres and 1% Deformed Steel (DS) fibres. The second combination included 1.5% MS fibres and 1.5% Hooked-end Steel (HS) fibres. The third combination included 1% DS fibres and 1.5% HS fibres. It was observed that the addition of hybrid steel fibres improved the strength and ductility of HSC compared to the reference specimens. It was observed that the addition of hybrid steel fibres into HSC increased the average strain corresponding to the peak stress by 19%, 21% and 48% for MD, DH, and MH specimen, respectively. However, the average peak stresses of MD and DH specimens were slightly decreased, while the average peak stress was increased by 8% for specimen MH compared to reference specimens. It was found that the inclusion of hybrid steel fibres into HSC led to significantly increasing in the ductility of HSC. The ductility was increased by 61%, 109% and 180% for MD specimens, Group DH and specimen MH, respectively compared to the ductility of reference specimens. In addition, the combination of the MS and HS fibres provided the highest enhancement in terms of strength and ductility compared to other combinations of the hybrid steel fibres and reference specimens.
Adding fibres into a concrete mixture leads to improvement of the mechanical characteristics, such as compressive strength, tensile strength, flexural strength and toughness, which is due to cut off macro- and micro-cracks. From the review of the literature, it is noted that the research studies about the behaviour of hybrid fibre reinforced concrete are limited. Therefore, more experimental work is needed in this research area to understand their behaviour.

2.8 Summary

This chapter presented a brief review of the available research studies regarding to plain concrete, fibre reinforced concrete and hybrid fibre reinforced concrete. The compressive stress-strain behaviour of concrete and fibre reinforced concrete were also presented. The mechanical properties (compressive strength, splitting tensile strength and flexural strength) and tensile stress-strain behaviour of fibre reinforced concrete were also presented and discussed. Also, it was observed that all of the experimental and analytical studies proposed strength models such as (compressive strength, splitting tensile strength and flexural strength) and regression-based stress-strain models, which are based on a specific range of compressive strength of concrete. The next chapter presents the structural behaviour of fibre reinforced concrete columns.
3: OVERVIEW OF FIBRE REINFORCED CONCRETE COLUMNS

3.1 General

This chapter presents an overview of the fibre reinforced concrete columns. The effects of the type, volume content and aspect ratio of the fibres on the behaviour of fibre reinforced concrete columns are addressed. The mechanism of confining pressure provided by steel helix and fibres to the concrete are discussed. Also, the available compressive stress-strain models for HSC and steel fibre reinforced HSC columns are presented in this chapter.

3.2 Behaviour of Fibre Reinforced Concrete Columns

Several research studies have investigated the potential of using fibres to improve the behaviour of the Reinforced Concrete (RC) columns. A summary of these experiments is presented in the next sections.

3.2.1 Using Fibres in Normal Strength Concrete Columns

Craig et al. (1984) tested 36 steel fibre RC columns with a rectangular cross-section of 152 mm width, 114 mm depth and 1676 mm length under axial load and shear load. To cast the RC columns, an unconfined concrete compressive strength of 27 MPa was used. Hooked end steel fibre having a length of 30 mm and a diameter of 0.5 mm were used. The volume contents of the fibres were 0.75% and 1.5%. The columns were reinforced with four longitudinal steel bars having a diameter of 12 mm. However, the different spacing of steel tie bars of 4 mm diameter at 76 mm, 114 mm, 152 mm, and 229 mm were used. The study concluded that the ductility of the specimens was slightly
increased with increasing volume content of steel fibres. Furthermore, the failure behaviour of the columns changed from a sudden brittle type to a ductile bending type due to the addition of steel fibre into RC columns. The study also observed that the shear capacity increased from 15% to 40%.

Mangat and Azari (1985) studied the combined effect of steel fibre and steel stirrup reinforcement on the properties of RC columns. Fourteen columns with an unconfined concrete compressive strength of 28 MPa, 31 MPa and 37 MPa were tested. Square RC columns with a cross-section of 150 mm and length of 750 mm were reinforced with four longitudinal reinforcement bars of 12 mm diameter of high strength deformed steel, and reinforced with the 6 mm diameter of mild steel plain ties bars spaced at 125 mm, 187 mm and 375 mm. Steel fibres of 25 mm length and 0.4 mm diameter were added to the RC in three different fibre volume content of 0%, 1.5% and 3%. The study concluded that RC columns with the steel stirrups spaced at 125 mm had shown no influence of steel fibre on the load distribution of the RC columns, but the columns with the stirrup spacing of 187 mm and 375 mm showed a significant influence of steel fibre on the load distribution.

In addition, RC columns with the 3% by volume of steel fibre led to slightly greater load carrying capacity than the non-fibre RC column. It was noted that reduction in steel ties spacing results in the reduction of the effectiveness of steel fibres in distributing the axial loads. It was also found that the increased volume content of steel fibre in the RC columns led to increased energy absorption capacity. This indicated that the columns can be more ductile and economy by reducing the lateral reinforcement ratio and increasing the fibres volume content.
Yashiro et al. (1989) tested 33 short square RC columns of a cross-section of 250 mm and 750 mm length, cast with an unconfined concrete compressive strength of 25 MPa, reinforced with steel fibre and tested under combined axial and shear loads. Steel tie reinforcement spaced at 60 mm and 90 mm (two types of arrangements), and three different volumetric tie ratios of 0.56%, 0.85% and 1.28% were used. Steel fibres were added to the concrete mix in ratios of 0%, 1% and 2% by volume. The variables studied were the level of applied load rate of 0.1 mm/min, 0.2 mm/min, and 0.3 mm/min, the tie reinforcement ratio, and volume content of steel fibres. Tie strain was measured for different volume content of steel fibre. It was found that the increasing of the fibre volume content reduced the tie strain for the same load level. The authors also concluded that the occurrence of bending cracks, bending shear cracks and shear cracks in steel fibre RC columns cannot be prevented. However, the propagation of the cracks reduced. This indicated that by increasing the fibre volume content resulted in more fibres being available to bear the applied load before transferring the load to the ties. Therefore the strain in steel ties was reduced with increasing fibre volume content.

Ganesan and Murthy (1990) tested eight square RC columns of a cross-section of 200 mm and length of 1000 mm under monotonic concentric load. Of these eight RC columns, four RC columns were reinforced with 1.5% by volume of steel fibres having an aspect ratio of 70. These columns were reinforced with eight longitudinal bars of 12 mm diameter and tie bars of 6 mm diameter with a diamond tie arrangement. A concrete cover of 25 mm was provided. The steel ties were spaced at 60 mm, 90 mm, 180 mm and 240 mm. For other four RC columns without steel fibres, cover spalling was evident with an increasing axial load. Except for the column with 240 mm spaced ties, no cover spalling was observed for the fibre RC columns with 90 mm and 180 mm spaced ties.
The authors observed that an increase in a volumetric ratio of tie reinforcement led to increasing the ultimate load and corresponding strain of fibre and non-fibre RC columns. Whereas, the percentage increase in the ultimate load and the corresponding strain was higher for fibre RC columns compared to non-fibre RC columns for the same tie detail.

Essawy and El-Hawary (1998) studied the effect of steel fibre and helical reinforcement on the strength and the ductility capacity of short rectangular RC columns subjected to a concentric load. A total of 36 short rectangular cross-section columns were constructed with an unconfined concrete compressive strength of 20 MPa. Columns were reinforced with steel helical reinforcement ratio ranged from 0.09% to 0.36%. A volumetric ratio of longitudinal steel reinforcement of 1.8% was used. The RC columns were reinforced with 4% by volume of round wire steel fibres having a diameter of 0.5 mm and length of 30 mm. It was found that the addition of steel fibres in RC columns led to a significant improvement in the ductility capacity. It was also noted that the load carrying capacity decreased with the inclusion of 4% by volume of steel fibres. The authors found that 4% by volume of steel fibre was not recommended, due to a reduction in workability of the fresh concrete which led to the reduction in load carrying capacity.

Lee (2007) tested eight steel fibre RC columns with a square cross-section of 250 mm and length of 900 mm under shear load to study the effect of steel fibres on shear strength. The longitudinal reinforcement ratio used was 1.2%, whereas, the shear reinforcement ratio was 0.26% and 0.21%. Hooked end steel fibres were used with the volume content of 1%, 1.5%, and 2% and aspect ratio of 60. The author concluded that
the inclusion of steel fibres into RC columns led to improvement in the strength and the ductility capacity more than an improvement in stiffness. It was also reported that 1.5% by volume content of steel fibre ensured maximum enhancement of both shear strength and ductility.

Germano *et al.* (2013) studied the influence of steel fibre and stirrups spacing on the flexural performance of RC columns tested under bending load about a section diagonal (bi-axial load) and bending load about a principal axis (mono-axial load). Sixteen RC columns were tested with the unconfined concrete compressive strength of 50 MPa. The columns with a square cross-section of 300 mm and length of 2400 mm were used. The RC columns were reinforced with eight longitudinal reinforcement of 16 mm diameter and 6 mm or 8 mm diameter transverse reinforcement, spaced at 80 mm or 100 mm. Hooked end steel fibre with 0.55 mm diameter and 35 mm length were added to the RC columns with a volume content of 1%. The authors concluded that the inclusion of steel fibre in the RC columns results in delaying premature cover spalling and bulking of the longitudinal reinforcement. They also reported that the addition of steel fibres appeared to be more noticeable in applying the mono-axial load by increasing ductility and the energy dissipation of the RC columns, while, the ductility and the energy dissipation was significantly decreased by applying biaxial load as compared with the non-fibre RC columns.

Palanivel and Sekar (2013) investigated the combined effect of polyolefin fibre volume content and spacing of lateral ties on RC prism subjected to axial compression. They tested 72 prisms of a square cross-section of 150 mm and length of 300 mm. The unconfined concrete compressive strength of 30 MPa was used. Polyolefin fibres having
0.79 mm diameter and 42 mm length were used. The volumetric ratio of polyolefin fibre for the columns tested were 0%, 0.3%, 0.5%, 0.7%, 0.9% and 1.2%. The RC prisms were reinforced with four longitudinal reinforcement bars of 4 mm diameter and 6 mm diameter of lateral ties bars spaced at 75 mm, 145 mm and 290 mm. Palanivel and Sekar (2013) indicated that early spalling of the concrete cover, strength and ductility improved with the inclusion of polyolefin fibres in the concrete mixture as polyolefin fibre provided indirect confinement to the concrete in axial compression. They also observed that the peak- and post-peak strains of RC prism were significantly larger than those with non-fibre RC prisms.

Osorio et al. (2014) demonstrated the seismic performance of synthetic fibre RC circular columns tested under combined constant axial load and reversed cyclic flexure load. Six circular cross-section columns of 305 mm diameter and a total length of 2000 mm, including 500 mm height of the base stub. The columns were reinforced with six longitudinal reinforcement bars of 20 mm diameter and hoop reinforcement of 10 mm diameter spaced at 42 mm, 75 mm and 100 mm. The unconfined concrete compressive strength was ranged from 30.8 MPa to 38.3 MPa. Synthetic fibres having an aspect ratio of 74 were added to the RC in a ratio of 1% by volume. It was concluded that the inclusion of 1% by volume of synthetic fibre into RC columns resulted in an increase in ductility by approximately 30% compared with non-fibre RC columns. The increase in energy dissipation capacities of 0%, 68% and 125% for synthetic fibre RC columns with lateral ties spaced at 42 mm, 75 mm and 100 mm, respectively, compared to non-fibre RC columns was observed. This indicated that an increase in tie reinforcement ratio led to decrease in energy dissipation capacity, and there was no more positive effect of fibres on the behaviour of the columns. It was also noted that the effectiveness
of fibre was decreased with an increasing hoop reinforcement ratio. However, with the same hoop reinforcement ratio, the inclusion of fibre into concrete column resulted in improvement in the post-peak response of the columns.

3.2.2 Using Fibres in High Strength Concrete Columns

Hsu et al. (1995) investigated behaviour of High Strength Concrete (HSC) slender columns with and without steel fibre subjected to combined axial compression and biaxial bending. Fourteen slender columns with a square cross-section of 76 mm and length of 1220 mm were tested. Unconfined concrete compressive strength was ranged from 72 MPa to 84 MPa. Concrete columns were tested with the static loading applied at different eccentricities on both x- and y-axes. Hooked end steel fibres with the volume content of 0.5% and 1% and an aspect ratio of 60 were added to the concrete columns. The columns were reinforced with four longitudinal reinforcement of 6 mm diameter or 4 longitudinal reinforcement of 10 mm diameter and 12.7 mm diameter for tie reinforcement spaced at 76 mm with 13 mm cover thickness.

Hsu et al. (1995) observed that the decreasing of the tie reinforcement spacing and adding steel fibre did not significantly affect the load carrying capacity. However, the crack length and crack zone were reduced. This led to the improved the ductility of concrete columns. Moreover, they modified an existing computer program to characterise the stress-strain behaviour of the slender columns. It was found that the computer analysis had slightly overestimated the descending portion of the load–deflection curves. In general, the theoretical load–deflection curves fit well with the experimental results.
Nagarajah and Sanders (1996) tested three RC columns (A1, A2, and A3) of a square cross section of 305 mm with footings. The columns were tested under a constant axial load equal to 20% of the ultimate load and with a cyclic lateral loading. Unconfined concrete compressive strength ranged from 79 MPa to 84 MPa. Crimped steel fibres having an aspect ratio of 70 and with volumetric ratio content of 0%, 0.75% and 1.25% were used in RC columns of A1, A2 and A3, respectively. All columns were reinforced with eight longitudinal reinforcement of 19 mm diameter. Ties of diameter 10 mm confinement were spaced at 76 mm, 76 mm, and 102 mm for RC columns A1, A2 and A3, respectively.

Nagarajah and Sanders (1996) found that the plastic hinge length for column A1, cast with HSC was approximately 250 mm; however, for columns A2 and A3, cast with fibre reinforced HSC, were about 200 mm. It was found that the addition of crimped steel fibres led to the reduction in the plastic hinge length. Moreover, for the A2 and A3 columns reinforced with steel fibres, the amount of cover spalling was much less as compared to column A1 without steel fibres. Finally, columns A1 and A2, with 76 mm tie spacing, failed by rupture of the longitudinal bars after crushing of the concrete. However, column A3, with 102 mm diameter ties spacing failed by buckling of the longitudinal bars.

Foster and Attard (2001) tested 21 steel fibre reinforced HSC columns under a static load applied at initial eccentricities of 0, 5, 8, 10, 20, 30 and 50 mm. Columns had square cross-sections of 155 mm and 200 mm, with the length of 1450 mm and 1820 mm, respectively. Unconfined concrete compressive strength ranged from 67 MPa to 88 MPa and concrete contained 2% (by weight) of hooked end steel fibres with an aspect
ratio of 84. The ties were spaced at 50 mm or 100 mm. The columns were cast in three series; series A had 155 mm square cross-section cast with a nominal unconfined concrete compressive strength of 90 MPa and eight longitudinal bars of 12 mm diameter. Series G and S columns also had 200 mm square cross-section and were cast with a nominal unconfined concrete compressive strength of 70 MPa and eight longitudinal bars of 12 mm diameter. For series A columns rectangular ties were used, while diamond tie arrangement was used for series G and S columns.

Foster and Attard (2001) compared tie strains at 90% of peak load ($P_u$) with tie strains at $P_u$ for the concentrically loaded specimens. It was found that minor strain development occurred in the ties while the columns were subjected to a relatively high axial load. This indicated that micro-cracking in the concrete core was controlled by the fibres. The research study showed that steel fibre reinforced HSC columns had superior ductility compared to non-fibre reinforced HSC columns. Furthermore, no cover spalling was observed.

Sarkar and Rangan (2001) tested six concrete columns cast with polyolefin fibre reinforced HSC, with the unconfined concrete compressive strength of 62 MPa, under different eccentricities in single and double curvature. All columns had a square cross-section of 175 mm and length of 1500 mm reinforced with eight longitudinal reinforcement bars of 12 mm diameter and 6 mm diameter of steel wire ties spaced at 100 mm. The polyolefin fibres with 25 mm length, 0.38 mm diameter and 1.1% by volume content of polyolefin fibre were added. It was observed that the experimental load capacity showed a good agreement with the predicted capacity and no cover spalling was evident.
Lima and Giongo (2004) tested 26 RC square columns under monotonically increasing concentric axial compression. All columns had a square cross-section of 150 mm with 500 mm length. Unconfined concrete compressive strength was between 68 MPa and 91 MPa. Hooked end steel fibres with an aspect ratio of 80 and volume contents of 0%, 0.5% and 1% of steel fibres were used. The columns were reinforced with four longitudinal bars of 12.5 mm diameter and steel tie bars of 6.3 mm diameter spaced at 50 mm and 150 mm.

Lima and Giongo (2004) observed that steel fibre prevented early concrete cover spalling. As a result of this, the buckling of the longitudinal reinforcement bars was delayed, even for columns with 150 mm tie spacing. The authors also found that the coefficient $\alpha_2$, that correlated the compressive strengths of plain concrete in a member and those obtained from standard cylinder test, is significantly influenced by the concrete strength, volumetric ratio of reinforcement and the volumetric fraction of steel fibre.

Hadi (2005) investigated the effect of including polypropylene fibre into HSC columns, and in particular in the concrete cover of columns. Unconfined concrete compressive strength ranged from 62.2 MPa to 65.1 MPa. Seven circular polypropylene fibre reinforced HSC columns of 205 mm diameter and 925 mm length of were tested under concentric load. Of these seven columns, one column had no fibre, two had fibres throughout the cross section, the other two had the fibre placed in the outer 45 mm on the cover of the columns, and the another two had fibres placed in the outer 22.5 mm on the cover of the columns. The columns were reinforced with six longitudinal bars of 12 mm diameter and 10 mm diameter of helical reinforcement spaced at 60 mm.
Polypropylene fibres, with volume content of 0.1% and 0.3%, were added throughout, and the outer cover of concrete columns.

Hadi (2005) observed that the location of the fibres did not significantly affect the load at which cover spalling took place. Also, the load carrying capacity of the fibre placed in the outer 22.5 mm was slightly lowered than the load capacity of the fibres placed in the outer 45 mm of the concrete cover. The author also observed that to enhance the ductility of the columns, it was more effective to place the fibre in the outer cover.

Sharma et al. (2007a) investigated the behaviour of steel fibre reinforced HSC short columns confined by square ties subjected to a concentric load. A total of 96 square columns of a cross-section of 150 mm and 600 mm length were cast. 24 unconfined and 72 confined specimens were tested. Unconfined concrete compressive strength ranged from 60 MPa to 80 MPa was used. These columns were reinforced with four longitudinal bars having a diameter of 12 mm and eight longitudinal bars having a diameter of 12 mm with yield tensile stress of 395 MPa, and two different grades (412 MPa and 520 MPa) of lateral reinforcement with 2.2%, 3.3%, and 5.6% were used. Flat crimped steel fibres were used with two aspect ratios (20 and 40), and three volume contents (1%, 1.5% and 2%). A small number of specimens using blended short and long fibres in equal weight at a total volume content of 1.5% were cast.

The study concluded that for a given tie confinement and concrete compressive strength, the strength and ductility of confined concrete strength were increased as the volume content of crimped fibres increased. In addition, the study reported that a delay in cover spalling occurred due to the inclusion of crimped fibres into HSC columns. The
study also indicated that the improvements in both strength and ductility can be optimised by a judicious blending of short and long crimped fibres, which may not be obtained by using a single aspect ratio of fibres. Finally, the study concluded that the HSC columns need either more fibre reinforcement for a given tie confinement or higher tie confinement for the same fibre content compared with lower concrete strength columns to achieve similar strength and ductility enhancements.

Djumbong et al. (2008) tested twelve circular concrete columns constructed with 1400 mm length and 300 mm diameter subject to concentric load. Of the total twelve columns, six columns were cast with the unconfined concrete compressive strength of 50 MPa and others six were cast with the unconfined concrete compressive strength of 80 MPa. All columns were reinforced with six longitudinal bars of 20 mm diameter and 6 mm steel helix spaced at 50 mm or 100 mm. Synthetic (polypropylene and polyethylene) fibre, with volume content of 0.5% and 1% were used. They observed that the use of 1% of synthetic fibres by volume led to increasing the load and corresponding strain of the columns at the first peak strength by 3% and 17%, respectively. However, the synthetic fibre had no significant effect on the post-peak behaviour of the concrete columns.

Hadi (2009) investigated the effect of inclusion of steel fibre into HSC columns, and in particular to the cover to the concrete columns under concentric load. Unconfined concrete compressive strength was ranged from 75.5 MPa to 90 MPa. Seven circular concrete columns of 205 mm diameter and 925 mm length were tested. Of these seven columns, one column was without fibres, three had fibres throughout the cross section, and three others had fibres placed only in the outer concrete. The columns were
reinforced with six longitudinal bars of 12 mm diameter and 10 mm diameter for helical reinforcement spaced at 60 mm. Enlarged end steel fibres, with an aspect ratio of 38, were added at 1%, 1.5% and 2% by volume to throughout, and in the outer cover of concrete columns. The results of the columns showed that the inclusion of enlarged end steel fibres into the cover of the column significantly increased its ductility. Furthermore, minimal cover spalling and very high strength could be obtained by adding 2% by volume of fibres into HSC columns.

Paultre et al. (2010) studied the performance of steel fibre reinforced HSC columns subject to concentric load. Twelve square cross-section concrete columns of 235 mm and 1400 mm length were tested. The unconfined concrete compressive strength of 100 MPa was used. The columns were reinforced with 2.2% and 3.6% of longitudinal reinforcement ratio, and 2.5%, 3.3% and 3.4% of lateral tie reinforcement ratio. The yield tensile stress for longitudinal bars was 400 MPa, and for the lateral reinforcement ranged from 400 MPa to 800 MPa. Crimped steel fibres, with an aspect ratio of 50 were added to the concrete in volume content of 0.25%, 0.5%, 0.75% and 1%.

Paultre et al. (2010) observed that the addition of steel fibre to the HSC concrete columns can prevent premature spalling of the concrete cover. Furthermore, the addition of crimped steel fibre into HSC columns increased both the axial strength and ductility. Moreover, Paultre et al. (2010) proposed the strength enhancement and strain enhancement models for steel fibre reinforced HSC by considering the geometric and mechanical parameters (volume content, aspect ratio, tensile strength) of the crimped steel fibres.
Khalil et al. (2012) tested fourteen concrete columns having a square cross-section of 100 mm and 1000 mm length in order to investigate the behaviour of HSC columns with and without steel fibres subjected to a concentric compression load. Unconfined concrete compressive strength ranged from 64.68 MPa to 78.86 MPa. The columns were reinforced with four longitudinal bars of 8, 10 or 12 mm diameter, and 8 mm diameter of lateral reinforcement spaced at 50 mm, 75 mm and 100 mm. Two types of hooked end steel fibres, with an aspect ratio of 100 and 60 and the volumetric ratio of 0.5% and 0.75% were used. They observed that the value of the compressive strength and the deformability of HSC columns with and without steel fibre were comparatively less marked as compared to the NSC columns with the same fibre volume content and aspect ratio. This means that HSC columns require more confinement than NSC to develop equivalent confinement pressures. They also found that the deformability of steel fibre reinforced HSC columns also increased as the aspect ratio of fibres increased.

3.3 Mechanism of Confinement Pressure Provided by Helix Reinforcement

Concrete under axial compression causes deformation (lateral expansion) in the lateral direction. This deformation develops due to Poisson’s ratio effect. In the initial stage of loading, when the axial strains are small, and therefore, the Poisson’s ratio effect of concrete is small, the lateral confinement provided by the helix reinforcement can be negligible. With an increase of the axial strain and the Poisson’s ratio of concrete, the lateral strain of concrete tends to increase. The concrete in the concrete core is restrained from lateral expansion by the helix reinforcement. The increasing lateral strain in the helix confinement results in increasing confining pressure applied to the concrete core.
The lateral confinement pressure provided by the helix reinforcement at maximum axial stress, $f_{l1}$, can be calculated as a function of the yield tensile stress of steel helix, area of the steel helix and volume of concrete core for a pitch or centre to centre spacing between two helix or hoop as follows:

\[
2 A_s f_y = d_c s' f_{l1} \quad \text{(3.1a)}
\]

\[
f_{l1} = \frac{2 A_s f_y}{d_c s'} \quad \text{(3.1b)}
\]

where $f_y$ is the yield tensile stress in the steel helix, $d_c$ is the diameter of concrete core, $s'$ is spacing between two helices from centre to centre, $A_s$ is area of the steel helix.

Figure 3.1 shows the confinement of the concrete core in circular column.

![Figure 3.1. Confined concrete circular column; (a) confinement of concrete core by steel helical and an isolated part of pitch height; (b) stress action on half-loop.](image)
Mander et al. (1988) developed the concept of the effective confined core area, which was earlier proposed by Sheikh and Uzumeri (1980), to determine effective confining pressure produced by steel hoop or helix. The concept is that the maximum lateral pressure from the steel hoop or helix can only be effectively developed on the concrete core where the confining pressure has fully developed due to arching action as shown in Figure 3.2. The study defined the effective lateral confining pressure as follow:

\[ f_l = f_{lx} k_e \]  
\[ k_e = \frac{A_e}{A_{cc}} \]

where \( k_e \) is confinement effectiveness coefficient, \( A_e \) is area of effectively confined concrete core, \( A_{cc} \) is calculated as \( A_c(1 - \rho_{cc}) \), \( \rho_{cc} \) is the ratio of area of longitudinal reinforcement to area of core of section, and \( A_c \) is the area of core of section enclosed by the centre lines of the perimeter hoop or helical.

In Figure 3.2, the arching action was assumed to occur in the form of a second-degree parabola with an initial tangent slope of 45°. Therefore, the area of an effectively confined concrete core at midway between the levels of transverse reinforcement and also the area of concrete core is presented as follows;

\[ A_e = \frac{\pi}{4} \left( d_c - \frac{s'}{2} \right)^2 \]  
\[ A_{cc} = \frac{\pi}{4} d_c^2 (1 - \rho_{cc}) \]
Thus, the confinement effectiveness factor for circular steel hoop and helix reinforcements is represented in Equations 3.5 and 3.6, respectively and are expressed as follows;

\[ k_{e,\text{hoop}} = \frac{\left[1 - \frac{s'}{2d_c}\right]^2}{1 - \rho_{cc}} \]  \hspace{1cm} (3.5)

\[ k_{e,\text{helix}} = \frac{\left[1 - \frac{s'}{2d_c}\right]}{1 - \rho_{cc}} \] \hspace{1cm} (3.6)

Thus, the effective lateral confining pressure on the concrete core is expressed as follows:

\[ f_l = 0.5 k_e \rho_s f_y \] \hspace{1cm} (3.7)

where \( \rho_s \) is the volume of the lateral steel divided by the volume of the concrete core; \( k_e \) for the hoop and helix reinforcement which were given in Equations 3.5 and 3.6, respectively.
Razvi and Saatcioglu (1999) proposed regression-based models of strength and strain enhancements of confined concrete applicable to the circular, square and rectangular columns reinforced with steel helix, rectangular reinforcements and welded fabrics. The proposed models are applicable for both normal and high strength concrete columns. These models considered the equivalent uniform lateral confinement pressure, which was determined from the sectional and material properties. The parameters incorporated in the model were the volumetric ratio, spacing, yield tensile stress, the arrangement of the transverse reinforcements, the distribution and amount of longitudinal steel, concrete compressive strength, and section geometry. The confining pressure relationship proposed by Razvi and Saatcioglu (1999) for the confined concrete strength enhancement is expressed as follows:
\[ f'_{ct} = f'_c + k_1 f_{le} \]  
(3.8a)

\[ k_1 = 6.7 (f_{le})^{-0.17} \]  
(3.8b)

\[ f_{le} = k_2 f_i \]  
(3.8c)

\[ k_2 = 0.26 \sqrt{\left( \frac{b_c}{s} \right) \left( \frac{b_c}{s_l} \right) \left( \frac{b_c}{f_i} \right)} \leq 1.0 \]  
(3.8d)

\[ f_i = \frac{\sum_{i=1}^{n} (A_s f_y \sin \alpha_i)}{s d_c} \]  
(3.8e)

where \( b_c \) refers to core dimension measured centre-to-centre of perimeter hoop (in mm); \( s \) refers to spacing of transverse reinforcement in longitudinal direction (in mm); \( s_l \) refers to spacing of longitudinal reinforcement, laterally supported by the corner of a hoop or the hook of a cross tie (in mm);

The confining pressure relationship proposed by Razvi and Saatcioglu (1999) was used to determine strain corresponding to maximum compressive stress in confined concrete columns as follows:

\[ \varepsilon'_{cc} = \varepsilon'_c (1 + 5K) \]  
(3.9a)

\[ K = \frac{k_1 f_{le}}{f'_c} \]  
(3.9b)

\[ k_1 = 6.7 (f_{le})^{-0.17} \]  
(3.9c)

### 3.4 Mechanism of Confinement Pressure Provided by the Steel Fibres

The inclusion of steel fibres into RC columns can provide additional confinement to the concrete core by delaying early cover spalling due to improvement in tensile resistance.
at cover concrete core interface. It can also reduce the strain on the lateral reinforcement by reducing the expansion in the lateral direction (Sharma et al. 2007b). Paultre et al. (2010) proposed strength enhancement and strain enhancement models to calculate confinement pressure provided by the steel fibre. Paultre et al. (2010) considered the influence of the mechanical properties of steel fibres, such as volume content, tensile strength and aspect ratio of the fibre, on the stress-strain behaviour of HSC square columns confined by lateral ties. The confinement pressure provided by steel fibres was defined as a confinement pressure developed at the yielding stage of the steel fibres by the action of fibre in an axial load and expressed as follows:

\[ f_b = \eta_\theta \tau_{fu} v_f (l_f/d_f) \]  

(3.10)

where \( \eta_\theta \) is fibre orientation factor. This factor was derived to account for the random orientation of the fibres in two- or three-dimensions by integrating the fibre load over all the possible fibre orientations. However, there is a great degree of disparity between the orientation factors due to different assumptions made and calculation techniques used, as can be seen in Table 3.1. Table 3.1 shows that it is difficult to determine or control the fibre orientation in an actual specimen; \( v_f \) is the volume content of the fibre; \( (l_f/d_f) \) is the aspect ratio of the fibre; where \( l_f \) is a length of fibre, \( d_f \) is a diameter of the fibre, and the bond shear strength of fibre is \( \tau_{fu} = 0.6 (f'_c)^{2/3} \) (Marti et al. 1999).
Table 3.1. Different fibre orientation factors by assuming 3D random orientation.

<table>
<thead>
<tr>
<th>References</th>
<th>Factors of fibre orientation ($\eta_\theta$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Romualdi and Mandel (1964)</td>
<td>0.405</td>
</tr>
<tr>
<td>Parimi et al. (1973)</td>
<td>0.637</td>
</tr>
<tr>
<td>Aveston and Kelly (1973)</td>
<td>0.5</td>
</tr>
<tr>
<td>Pakotiprapha (1976)</td>
<td>0.25</td>
</tr>
<tr>
<td>Foster (2001)</td>
<td>0.375</td>
</tr>
<tr>
<td>Voo and Foster (2003)</td>
<td>Variable ($\leq$0.5)</td>
</tr>
</tbody>
</table>

3.5 Stress-Strain Relationship of Confined HSC Columns

The analysis of structural members requires a full stress-strain relationship of concrete in compression for both confined and unconfined states. Established stress-strain models for confined NSC are based on extensive experimental data. These models are based on the test results of NSC columns. However, these models could be inadequate for confined HSC columns, which possess reduced ductility in the stress-strain response. Thus, the NSC models if applied to HSC could be overestimated the ductility. As a result, a number of stress-strain models specifically for confined HSC columns have been proposed. The stress-strain relationships of the unconfined HSC with and without steel fibres are discussed in Chapter 2. In following sections, the stress-strain for the confined HSC columns with and without steel fibre are presented and discussed.

3.5.1 Model Proposed by Cusson and Paultre (1995)

Cusson and Paultre (1995) developed a confinement model for the HSC based on test results of 50 large-scale HSC tied square columns tested under concentric load. 30 HSC tied square cross-section columns of 235 mm and length of 1400 mm were tested by the authors and twenty HSC tied square cross-section columns of 225 mm and length of
715 mm were tested by Nagashima et al. (1992). Unconfined concrete compressive strength ranged from 60 MPa to 120 MPa. The yield tensile stress of ties ranged from 400 MPa to 800 MPa. The proposed model considered tie yield tensile stress, tie configuration, lateral reinforcement ratio, lateral spacing, and longitudinal reinforcement ratio. It was found that the amount of transverse reinforcement was the most important parameter that influenced the stress-strain relationship of confined concrete. Cusson and Paultre (1995) proposed analytical models of the strength and strain enhancements of confined HSC columns using regression analyses as shown in Table 3.2. Cusson and Paultre (1995) indicated that an increasing in the tie yield tensile stress can only be effective in strength enhancement for well-confined columns with a large amount of transverse reinforcement. An ascending branch of the stress-strain relationship was proposed based on a relationship originally proposed by Popovics (1973) as shown in Table 2.2, but the parameters of the model were recalibrated based on a large number of test data collected by the authors. A descending branch of the stress-strain curve is a modification of the relationship proposed by Fafitis and Shah (1985) for confined HSC as follows:

\[ f_c = f_{cc}' \cdot \exp[k_1(\varepsilon_c - \varepsilon_{cc}')^k_2] \]  \hspace{1cm} (3.11)

Cusson and Paultre (1995) introduced an effective confinement index at the peak confined HSC stress and is expressed as follows:

\[ I'_e = f_t/f_{cc}' \]  \hspace{1cm} (3.12)
3.5.2 Model Proposed by Razvi and Saatcioglu (1999)

Razvi and Saatcioglu (1999) proposed confinement models for confined NSC and HSC columns using their extensive test data as well as the experimental results of other research studies. These included the test results of nearly full-size specimens of different shapes, sizes, and reinforcement configurations, tie yield tensile stress ranged from 400 MPa to 1387 MPa, and unconfined concrete compressive strength ranged from 30 MPa to 130 MPa. It was found that the strength enhancement of concrete due to confinement produced by steel helical reinforcement did not change with concrete compressive strength. Thus, it was suggested that if the same percentage of strength enhancement is desired, HSC columns are required to have proportionately higher confinement than those confined with NSC columns. The strength and strain enhancements ratio models of HSC columns were proposed based on regression analyses as shown in Table 3.2. Their stress-strain relationship is based on a relationship proposed by Popovics (1973); however, the parameters of the model were recalibrated to cover a wide range of the unconfined concrete compressive strength.

3.5.3 Model Proposed by Legeron and Paultre (2003)

Legeron and Paultre (2003) proposed confinement model for NSC and HSC columns based on a large number of test results of circular, square and rectangular columns tested under various experimental conditions (including studies undertaken by the authors and a number of other research studies). Unconfined concrete compressive strength ranged from 20 MPa to 140 MPa, and yield tensile stress of the lateral steel reinforcement ranged from 300 MPa to 1400 MPa. For HSC, Legeron and Paultre (2003) considered the fact that the lateral steel might not yield at the peak stress of the
confined concrete. Thus, they have incorporated this fact into respective confinement models by proposing an expression to determine the stress in the lateral steel reinforcement, $f_h$, at the peak of the confined concrete stress-strain response, and are expressed as follows:

$$f_h = \begin{cases} f_{hy} & \text{if } \kappa \leq 10 \\ 0.25 \frac{f'_c}{\rho_{sey}(\kappa - 10)} \geq 0.43 \frac{\varepsilon'_c E_s}{\varepsilon'_{hy}} & \text{if } \kappa > 10 \end{cases}$$  \hspace{1cm} (3.13a)

where $\kappa$ is the parameter used to determine the yield tensile stress of the lateral steel reinforcement occurring at the peak strength of confined concrete, and is expressed as follows:

$$\kappa = \frac{f'_c}{\rho_{sey} E_s \varepsilon'_c}$$  \hspace{1cm} (3.13b)

Legeron and Paultr (2003) also proposed strength and strain enhancements models based on the regression analyses as given in Table 3.2.

3.6 Stress-Strain Relationship for Steel Fibre Reinforced HSC Confined Columns

Numerous research studies proposed compressive stress-strain models for unconfined SFRC. The compressive stress-strain models of the unconfined SFRC are presented and discussed in Chapter 2. The available stress-strain models for confined steel fibre RC columns are quite limited. The following sections presented the available models.
3.6.1 Model Proposed by Paultre et al. (2010)

Paultre et al. (2010) proposed confinement model for Steel Fibre Reinforced High Strength Concrete (SFR-HSC) square columns based on the experimental results of 15 large-scale SFR-HSC under concentric load. Unconfined concrete compressive strength ranged from 88 MPa to 101 MPa, and yield tensile stress of steel ties ranged from 392 MPa to 856 MPa. In the proposed model, the parameterise such as tie yield strength, tie reinforcement ratio, tie spacing, longitudinal reinforcement ratio, and the mechanical and geometrical properties (volume content, aspect ratio and tensile strength) of the steel fibres were considered. Paultre et al. (2010) used the same expression that was proposed by Cusson and Paultre (1995) to find that the effective confinement index at the peak stress of the SFR-HSC square columns and is expressed as follows:

\[ I'_e = \frac{f_t}{f'_c} + \frac{f_b}{f'_c} \]  

(3.14)

where \( f_{te} \) is the effective lateral confinement pressure, \( f_b \) is the confinement pressure provided by the steel fibres which can be determined by using Equation 3.10, and \( f'_c \) is the unconfined compressive strength of HSC. Furthermore, the authors developed an expression to determine the stress in the lateral reinforcement at the peak stress in steel fibre reinforced concrete columns, \( f_h \), based on Equation 3.13a as follows:

\[
 f_h = \begin{cases} 
 f_{hy} & \text{if } \kappa \leq 10 \\
 0.25 f'_c + 10\eta_\sigma\tau_{tu}v_t \left( \frac{l_f}{d_t} \right) \rho_{sey}(\kappa - 10) & \geq 0.43 \epsilon'_c E_s \neq f_{hy} \text{ if } \kappa > 10 
\end{cases}
\]  

(3.15)
Paultre et al. (2010) also adopted the expression of the confined concrete compressive strength and corresponding strain proposed by Cusson and Paultre (1995) to determine the strength and strain enhancements of SFR-HSC square columns as given in Table 3.2. Finally, the ascending branch of the stress-strain relationship is based on a relationship proposed by Popovics (1973); however, the parameters of the model were recalibrated. The descending branch of the stress-strain curve is a modification of the relationship proposed by Fafitis and Shah (1985) for confined HSC.

3.6.2 Model Proposed by Balanji et al. (2015)

Balanji et al. (2015) proposed models for predicting the confined concrete compressive strength of Steel Fibre Reinforced Normal Strength Concrete (SFR-NSC) and SFR-HSC square columns. The proposed model was based on a total of 71 experimental investigation databases of SFR-NSC and SFR-HSC square columns have been collected from available research studies. Unconfined concrete compressive strength ranged from 43 MPa to 101 MPa. In the proposed model, the parametrise such as the effect of unconfined concrete compressive strength, confinement pressure provided by the lateral reinforcement and confinement pressure provided by the steel fibres were considered. Balanji et al. (2015) used artificial neural network analysis to predict the compressive strength model; further details are presented and discussed in Chapter 4. The authors found that the prediction results of the developed ANN compressive strength models fit very well with the experimental results.
Table 3.2. Existing models of strength and strain enhancements of the confined HSC columns with and without steel fibres reinforcement.

<table>
<thead>
<tr>
<th>References</th>
<th>Strength enhancement</th>
<th>Strain enhancement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Strain corresponding to the peak stress</td>
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<tr>
<td><strong>Without steel fibres reinforcement</strong></td>
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<td></td>
</tr>
<tr>
<td>Cusson and Paultre (1995)</td>
<td>$f_{cc}' / f_c' = 1 + 2.1 \left( \frac{f_l}{f_c} \right)^{0.7}$</td>
<td>$\frac{\varepsilon_{cc}'}{\varepsilon_c'} = 1 + 0.21 \left( \frac{f_l}{f_c} \right)^{1.7}$</td>
</tr>
<tr>
<td>Razvi and Saatcioglu (1999)</td>
<td>$f_{cc}' / f_c' = 1 + k_1 \frac{f_l}{f_c}$</td>
<td>$\frac{\varepsilon_{cc}'}{\varepsilon_c'} = 1 + 5k_3 \left( \frac{k_1 f_l}{f_c} \right)$</td>
</tr>
<tr>
<td>Legeron and Paultre (2003)</td>
<td>$f_{cc}' / f_c' = 1 + 2.4 \left( \frac{f_l}{f_c} \right)^{0.7}$</td>
<td>$\frac{\varepsilon_{cc}'}{\varepsilon_c'} = 1 + 35 \left( \frac{f_l}{f_c} \right)^{1.2}$</td>
</tr>
<tr>
<td><strong>With steel fibres reinforcement</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Paultre et al. (2010)</td>
<td>$f_{cc}' / f_c' = 1 + 2.1 \left( \frac{f_l}{f_c} + \frac{f_b}{f_c} \right)^{0.7}$</td>
<td>$\frac{\varepsilon_{cc}'}{\varepsilon_c'} = 1 + 0.21 \left( \frac{f_l}{f_c} + \frac{f_b}{f_c} \right)^{1.7}$</td>
</tr>
</tbody>
</table>
3.7 Summary

From the review of the literature, it was found that the addition of fibres can significantly delay the cover spalling and improve the ductility. The review of the literature has shown that the majority of the existing studies of steel RC columns were related to square reinforced concrete columns, and very limited studies have been conducted on circular reinforced concrete columns. Also, it was found that the majority of the research studies investigated the behaviour of RC columns with and without steel fibre under concentric axial load. However, only a few of the research studies investigated the behaviour of the columns under eccentric axial load. The behaviour of the RC columns with different types of steel fibre tested under eccentricity is still not understood. Furthermore, no study was conducted on the behaviour of hybrid fibre RC column under eccentric axial load. Thus, a gap exists in the literature on the behaviour of the different type of steel fibre, and hybrid fibre reinforced HSC circular columns subjected to the concentric axial load, eccentric axial load.

In the next chapter, the Artificial Neural Network (ANN) analysis is used to develop ANN compressive strength model and ANN splitting tensile strength model of SFRC. Also, the ANN is used to propose compressive strength of SFR-NSC and SFR-HSC square columns.
4: PROPOSED STRENGTH MODELS USING ARTIFICIAL NEURAL NETWORK ANALYSIS

4.1 General

In this chapter, Artificial Neural Network (ANN) analysis is used to predict the compressive strength and splitting tensile strength models of Steel Fibre Reinforced Concrete (SFRC). The experimental investigation databases of 102 SFRC cylinders tested under compression load and splitting tensile load were compiled and analysed. These databases were used to develop ANN compressive strength model and ANN splitting tensile strength model as a function of the compressive strength of concrete, volume content, aspect ratio and nominal yield tensile stress of steel fibres.

The ANN analysis is also used to predict compressive strength of Steel Fibre Reinforced Normal Strength Concrete (SFR-NSC) and Steel Fibre Reinforced High Strength Concrete (SFR-HSC) square columns. The experimental investigation databases of 71 SFR-NSC and SFR-HSC square columns tested under concentric axial load were compiled and analysed. These databases were used to develop ANN compressive strength model as a function of the compressive strength of concrete, confinement pressure provided by lateral reinforcement and confinement pressure provided by steel fibres. A statistic performance of the proposed models is presented. The following sections present and discuss the details of ANN analysis.
4.2 Artificial Neural Network Analysis

The Artificial Neural Network (ANN) is an analysis toolbox available in MATLAB (2013b). ANN can be used for pattern recognition, model prediction and data classification. The ANN analysis consists of input, target (experimental output), weight factor, bias, and transfer functions. Figure 4.1 presents a schematic diagram of ANN. The ANN output is generated in two layers. In the first layer, *i.e.*, a hidden layer, the input scalar \( I \) is multiplied by the weight scalar \( W \) of the hidden layer to form \( IW \) known as an input weight matrix, and a bias matrix \( b_1 \) of the hidden layer is added to \( IW \) to obtain an input function. The input function is subjected to transfer function, such as Linear (Purelin), Tan-Sigmoid (Tansig) and Log-Sigmoid (Logsig) functions to obtain an output scalar \( L \) of the hidden layer. In the second layer, *i.e.*, an output layer, the output scalar \( L \) from the hidden layer multiplied by the weight scalar \( W \) of the output layer to obtain an output weight matrix \( LW \), and a bias matrix \( b_2 \) of the output layer is added to the \( LW \) to obtain an output function. The output function is subjected to the transfer function to obtain an ANN output \( o \).

![Figure 4.1](image.png)

Figure 4.1. A schematic diagram of the component of the ANN (MATLAB 2013b).
4.2.1 General Steps for ANN Analysis

In order to conduct the ANN analysis, the following steps can be followed:

1. Select input data set and target (experimental output) data set.

2. Determine numbers of the neurons for the hidden and the output layers.

3. Select the architecture of the neural network that is available in the ANN toolbox such as Cascade-forward back propagation and Feed-forward back propagation.

4. Select the training function, *i.e.*, Levenberg Marquardt (LM), Bayesian Regularisation (BR), Scaled Conjugate Gradient (SCG), available in MATLAB (2013b) in order to train the neural network.

5. Divide the input data set, and target data set into data subset of training data, validation data and testing data.

6. Select the transfer function available in MATLAB (2013b) such as Purelin, Tansig and Logsig.

7. Obtain the weight factors, basis and output from the trained neural network.
4.2.2 Mathematical Derivations of the ANN Analysis

The ANN output of the output layer using normalized input and target, weight factors (input weight matrix \((IW)\) and output weight matrix \((LW)\)) of the neural network and biases \((b_1\) and \(b_2\)) of the neural network, is obtained using the Equation 4.1.

\[
y_1 = \text{Transfer function} \ (IW \times X_n + b_1) \tag{4.1a}
\]

\[
y = \text{Transfer function} \ (LW \times y_1 + b_2) \tag{4.1b}
\]

where, \(X_n\) is the input matrix, \(y_1\) is the output of the hidden layer, \(y\) is the ANN output. Two transfer functions \(i.e.,\) the Purelin and Tansig, are used in this study to train the neural network and are expressed as follows:

\[
\text{Purelin} \ (i) = i \tag{4.2}
\]

\[
\text{Tansig} \ (i) = \tanh \ (i) = \frac{1 - e^{-2i}}{1 + e^{-2i}} \tag{4.3}
\]

Pham and Hadi (2014) and Balanji et al. (2015) reported that the neural network models trained with the use of Tansig transfer function lead to predicting the ANN outputs with less errors compared to a neural network trained with the use of Purelin transfer function. However, the neural network trained with the use of Purelin transfer function lead to simplified ANN models while the neural network trained with the use of Tansig transfer function results in complicated ANN models.
The mathematical derivation of the neural network using normalized input and target and neural network weights and biases is expressed as follows:

\[
y = \sum_{i=1}^{i} \left( \frac{y_{\text{max}} - y_{\text{min}}}{x_{i \text{ max}} - x_{i \text{ min}}} \right) w_i x_i + \frac{(y_{\text{max}} + y_{\text{min}})}{2} + \frac{(y_{\text{max}} - y_{\text{min}})}{2} a
\]

\[
- \sum_{i=1}^{i} \left[ \left( \frac{y_{\text{max}} - y_{\text{min}}}{x_{i \text{ max}} - x_{i \text{ min}}} \right) w_i x_{i \text{ min}} + \frac{(y_{\text{max}} - y_{\text{min}})}{2} \times w_i \right]
\]  

(4.4)

where \( y \) is the ANN output, \( y_{\text{max}} \) is the normalized maximum target, \( y_{\text{min}} \) is the normalized minimum target, \( x_{i \text{ max}} \) is the normalized maximum input, \( x_{i \text{ min}} \) is the normalized minimum input and \( w_i \) is the weight. The Equation 4.4 can be simplified as follows:

\[
y = \sum_{i=1}^{i} k_i x_i + c \quad \text{(4.5a)}
\]

\[
k_i = \sum_{i=1}^{i} \left( \frac{y_{\text{max}} - y_{\text{min}}}{x_{i \text{ max}} - x_{i \text{ min}}} \right) w_i \quad \text{(4.5b)}
\]

\[
c = \frac{y_{\text{max}} + y_{\text{min}}}{2} + \frac{(y_{\text{max}} - y_{\text{min}})}{2} a
\]

\[
- \sum_{i=1}^{i} \left[ \left( \frac{y_{\text{max}} - y_{\text{min}}}{x_{i \text{ max}} - x_{i \text{ min}}} \right) w_i x_{i \text{ min}} + \frac{(y_{\text{max}} - y_{\text{min}})}{2} \times w_i \right]
\]

(4.5c)

\[
a = LW(b_1) + b_2 \quad \text{(4.5d)}
\]

where \( k_i \) is proportional factors.
4.3 Compressive and Splitting Tensile Strength Models of SFRC

The brittle behaviour of concrete can be reduced by the inclusion of steel fibres (Zollo 1997, Bentur and Mindess 2006, Holschemacher et al. 2010). Hence, it was observed that the strength of concrete such as compressive strength, tensile strength, shear strength, and impact resistance improve with the inclusion of steel fibres (Batson 1976, Banthia and Mindess 1996, Gao et al. 1997, Bentur and Mindess 2006, Altun et al. 2007, Aydin 2013). In addition, the compressive strength and tensile strength of SFRC are two mechanical characteristics in the design of SFRC. The tensile strength of SFRC can be determined by different tests, for example, direct tensile test, flexural and splitting tensile tests. A number of research studies have studied the strength behaviour of SFRC (Wafa and Ashour 1992, Khaloo and Kim 1996, Song and Hwang 2004, Bentur and Mindess 2006, Thomas and Ramaswamy 2007, Yazici et al. 2007, Ramadoss and Nagamani 2008, Al-Hassani et al. 2014, Balanji et al. 2016). In general, they found that the splitting tensile strength, shear strength, and flexural strength of SFRC increase with the increasing volume content and aspect ratio (length/diameter) of steel fibres.

The previous research studies proposed strength models of compressive strength and splitting tensile strength to determine the effectiveness of parameters, such as concrete compressive strength, volume content, and aspect ratio of steel fibres on the compressive strength and splitting tensile strength behaviour of SFRC. In general, it was found that the improvements in the strength of SFRC can be a function of the different parameters, such as concrete compressive strength, volume content, aspect ratio, and type of steel fibres. However, the available strength models are based on regression analyses of limited experimental investigation results. In addition, these models have been proposed based on a limited range of concrete compressive strength,
volume content and aspect ratio of steel fibres to develop strength models of SFRC as explained in Chapter 2. Table 2.2 presents the available strength models of SFRC. Furthermore, the available strength models were validated for a limited range of data points and for a specific type of steel fibres such as hooked-end and crimped steel fibres. It was found that the available strength models were in reasonable agreement with their experimental results. However, the errors estimating in the available strength models for a limited range of data points still exist. It was indicated from the literature review in Chapter 2 that there are different arguments for the use of parameters that effect to the strength of SFRC.

Recently, ANN analysis has been successfully applied in the modelling of various engineering problems. It provides a good alternative to statistical regression and numerical methods (Behnood et al. 2015). The majority of research studies have used maps of the ANN analysis to study their accuracy without providing any mathematical formulas (Ni and Wang 2000, Hadi 2003, Oreta and Kawashima 2003, Oztas et al. 2006, Sha and Edwards 2007, Bilim et al. 2009, Naderpour et al. 2010, Perera et al. 2010, Acikgenc et al. 2014, Behnood et al. 2015). A few of the research studies have produced mathematical formulas to show the accuracy of ANN analysis (Pham and Hadi 2014, Balanji et. al. 2015). In addition, most of the available research studies on ANN analysis have been designed with the multiple input parameters and only one output parameter, which generally were compressive strength of concrete (Ni and Wang 2000, Oreta and Kawashima 2003, Oztas et al. 2006, Altun et al. 2007, Naderpour et al. 2010, Perera et al. 2010, Pham and Hadi 2014, Behnood et al. 2015). However, a few of research studies on the ANN analysis have been designed with multiple input and outputs parameters (Topcu and Saridemir 2007, Topcu and Saridemir 2008, Acikgenc et al. 2014).
In this study, the compressive strength model and splitting tensile strength model of SFRC were proposed using ANN analysis. The proposed models cover a wide range of the experimental databases that collected from the literature.

4.3.1 Experimental Investigation Databases of SFRC

The experimental investigation databases of 102 SFRC were gathered from a number of research papers of Wafa and Ashour (1992), Khaloo and Kim (1996), Song and Hwang (2004), Thomas and Ramaswamy (2007), Ramadoss and Nagamani (2008), and Aydin (2013), Balanji et al. (2016). The experimental investigation databases included information of the compressive strength of concrete, the volume content of steel fibre, the aspect ratio of steel fibres, types of steel fibres, the nominal yield tensile stress of steel fibres, the compressive strength of SFRC, and splitting tensile strength of SFRC. The compressive strength test was conducted according to ASTM C 39 (ASTM 1992, 2011), and the splitting tensile strength test was conducted according to ASTM C 496 (ASTM 1990, 1999, 2011). Each data point in Wafa and Ashour (1992) study comprised of an average of 18 cylinders. However, for the other studies, each data point comprised of an average of three cylinders.

Table 4.1 presents the experimental investigation databases of SFRC. The databases cover a wide range of the data points. The compressive strength of the concrete used in the ANN analysis ranged between 24 MPa and 94 MPa. In addition, the volume content of steel fibres ranged between 0.25% and 4%, and aspect ratio of steel fibres ranged between 30 and 80. The nominal yield tensile stress of steel fibres ranged between 260 MPa and 2500 MPa. The results of the experimental investigation, i.e., compressive strength and splitting tensile strength were tested at 28 days. The experimental investigation databases included four types of steel fibres, i.e., straight, deformed, hooked end and crimped.
Table 4.1. Experimental investigation databases of SFRC.

<table>
<thead>
<tr>
<th>References</th>
<th>Compressive strength of the concrete ($f'_c$) (MPa)</th>
<th>Volume content ($v_f$%)</th>
<th>Aspect ratio ($l/d$)</th>
<th>Nominal yield tensile stress ($f_{ty}$) (MPa)</th>
<th>Compressive strength of SFRC (MPa)</th>
<th>Splitting tensile strength of SFRC (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Wafa and Ashour</strong></td>
<td>93.49</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<td>(1992)</td>
<td>93.49</td>
<td>H^1</td>
<td>0.25</td>
<td>75</td>
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<td>(1996)</td>
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Table 4.1. Continued

<p>| References | Compressive strength of the concrete ( (f'_c) ) (MPa) | Steel fibres | | Compressive strength of SFRC (MPa) | | Splitting tensile strength of SFRC (MPa) |
|---|---|---|---|---|---|
| Ramadoss and Nagamani (2008) | | | | | |
| | 46.85 | C | 1.50 | 80 | 910 | 52.68 | 6.04 |
| | 52.56 | - | - | - | - | 52.56 | 4.38 |
| | 52.56 | C | 0.50 | 80 | 910 | 54.77 | 5.48 |
| | 52.56 | C | 1.00 | 80 | 910 | 56.01 | 6.37 |
| | 52.56 | C | 1.50 | 80 | 910 | 57.40 | 6.83 |
| | 52.69 | - | - | - | - | 52.69 | 4.41 |
| | 52.69 | C | 0.50 | 80 | 910 | 55.64 | 5.69 |
| | 52.69 | C | 1.00 | 80 | 910 | 57.85 | 6.67 |
| | 52.69 | C | 1.50 | 80 | 910 | 58.23 | 6.73 |
| | 55.85 | - | - | - | - | 55.85 | 4.75 |
| | 55.85 | C | 0.50 | 80 | 910 | 59.65 | 5.94 |
| | 55.85 | C | 1.00 | 80 | 910 | 61.05 | 6.65 |
| | 55.85 | C | 1.50 | 80 | 910 | 61.44 | 7.26 |
| | 60.10 | - | - | - | - | 60.10 | 4.86 |
| | 60.10 | C | 0.50 | 80 | 910 | 62.81 | 6.35 |
| | 60.10 | C | 1.00 | 80 | 910 | 64.01 | 6.73 |
| | 60.10 | C | 1.50 | 80 | 910 | 64.56 | 7.15 |
| | 63.86 | - | - | - | - | 63.86 | 5.12 |
| | 63.86 | C | 0.50 | 80 | 910 | 67.12 | 6.35 |
| | 63.86 | C | 1.00 | 80 | 910 | 68.91 | 7.18 |</p>
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Note: H$^+$ refers to Hooked end steel fibre, C$^+$ refers to Crimped steel fibre, S$^+$ refers to Straight steel fibre, D$^4$ refers to Deformed steel fibre.
The compressive strength of concrete \( (f_c') \), volume content of steel fibres \( (v_t\%) \), aspect ratio of steel fibres \( (l/d) \) and nominal yield tensile stress \( (f_{ty}) \) were considered as input parameters. Figure 4.2 presents the range of the compressive strength of concrete data points. The compressive strength of concrete varies from 24 MPa to 94 MPa with 76% of the data points with compressive strength in the range of 50 MPa - 94 MPa. Figure 4.3 presents the range of the volume content of steel fibres data points. The volume content of steel fibres varies from 0.25% to 4% with 63% of the data points with volume content in the range of 0.25% - 1%. This indicates that the most of the previous studies have used 1% or less by volume of steel fibres. The reason for this could be the reduction of the workability of SFRC with an increasing the volume content of steel fibres. Figure 4.4 presents the range of the aspect ratio of steel fibres data points. The aspect ratio of steel fibres varies from 30 to 80 with 59% of the data points with aspect ratio in the range of 30 - 60. Figure 4.5 presents the range of the nominal yield tensile stress of steel fibres data points. The nominal yield tensile stress of steel fibres varies from 260 MPa to 2500 MPa with 65% of the data points with nominal yield tensile stress in the range of 260 MPa – 1000 MPa. In addition, the data points of 56% have hooked end types of steel fibre, whereas, 29% of the data points have crimped types of steel fibres, and only 7.5% of the data points have straight and deformed steel fibres as shown in Figure 4.6.

The results of the experimental databases including compressive strength and splitting tensile strength of SFRC were used as output parameters. Figure 4.7 presents the range of the compressive strength of SFRC \( (f_{cd}) \) data points. The compressive strength of SFRC varies from 24 MPa to 98 MPa with 81% of the data points with compressive strength of SFRC in the range of 50 MPa – 98 MPa. Figure 4.8 presents the range of
splitting tensile strength of SFRC ($f_{spf}'$) data points. The splitting tensile strength of SFRC ($f_{spf}'$) varies from 2 MPa to 12 MPa with 76% of the data points with splitting tensile strength of SFRC in the range of 5 MPa – 12 MPa.

Figure 4.2. Frequency distribution of the compressive strength of the concrete.

Figure 4.3. Frequency distribution of the volume content of steel fibres.
Figure 4.4. Frequency distribution of the aspect ratio of steel fibres.

Figure 4.5. Frequency distribution of the nominal yield tensile stress of steel fibres.
Figure 4.6. Frequency distribution of the type of steel fibres.

Figure 4.7. Frequency distribution of the compressive strength of SFRC.
4.3.2 Design of ANN Compressive Strength and ANN Splitting Tensile Strength Models

In this study, ANN analysis was designed based on four input parameters including $f'_c$, $v_t\%$, $l/d$, and $f_{tf}$, and two output parameters (target) including $f'_{ct}$ and $f'_{spf}$. The optimum model of ANN architecture, such as the number of the hidden layers and the corresponding number of hidden nodes, needs to be determined. The use of one hidden layer and one output layer of neurons in the multilayered Feed-forward network can produce accurate approximations (Hornik et al. 1989), and this is what was utilized in this study. Based on trial and error approach, the optimal number of hidden nodes was obtained by training the network with a set of random initial weights and a fixed learning rate of 0.01. The determination of the number of hidden neurons, and output neurons results in the ability to estimate an appropriate number of samples for the
training data subset. Upadhyaya and Eryurek (1992) suggested the Equation 4.6 to estimate the necessary number of training samples:

\[
\frac{w}{o} \leq n \geq \frac{w}{o} \times \log_2 \frac{w}{o}
\]

(4.6)

where \( n \) is training sample numbers, \( w \) is weight numbers and \( o \) is an output parameter number. Using number of weights and the number of output parameters, Equation 4.6 was developed for ANN compressive strength model and ANN splitting tensile strength model, and is expressed as follows:

\[
60 \leq n = 72 \leq 354
\]

(4.7)

Once the network architecture has been designed, and the neural network is trained. To ensure statistical consistency of the data needed for the ANN model development, all input and target parameters were randomly divided into subset data of training data (70%), testing data (15%), and validation data (15%) (Pham and Hadi 2014). Training division was used to find the optimum value of the weight and biases of the network. Whereas, the testing division was used to compare different models and validation division was used to minimise the validation error of the network.

In order to train the neural network, the Levenberg-Marquardt (LM) algorithm training function was used. Although the LM algorithm needs more memory space than other
methods, however, it is a faster algorithm in the ANN toolbox (Pham and Hadi 2014). Thus, LM algorithm was used in this study.

A Feed-forward back propagation type of network was used with the four input neurons $f_c'$, $v_l\%$, $l/d$, and $f_{tf}$, ten neurons in the hidden layer, and two neurons in the output layer of $f_{cf}'$ and $f_{spf}'$ of SFRC were used. In addition, the Trainlm function of training, the Learngdm adoption learning function, the MSE performance function and the Pureline transfer function in both the hidden and the output layers were used. The network architecture of the proposed ANN $f_{cf}'$ and $f_{spf}'$ models is presented in Figure 4.9.

![Network Architecture Diagram](image)

Figure 4.9. The network architecture of the proposed ANN $f_{cf}'$ and $f_{spf}'$ models.
4.3.3 Proposed ANN Compressive Strength and ANN Splitting Tensile Strength Models of SFRC

In this study, two strength models were developed based on the experimental investigation databases. In order to calculate ANN compressive strength model and ANN splitting tensile strength model of SFRC, simple equations were developed as a function of the $f_c'$, $v_t\%$, $l/d$, and $f_{tf}$. After training ANN analysis, the input weight matrix ($IW$), the output weight matrix ($LW$), and the bias matrix ($b_1$ and $b_2$) were obtained. The proposed ANN compressive strength model and ANN splitting tensile strength model are presented in Equation 4.8 and Equation 4.9, respectively.

$$f_{cf}' = \begin{bmatrix} k_c' \\ v_t'\% \\ l/d \\ f_{tf}' \end{bmatrix} + c_c$$  \hspace{1cm} (4.8a)

$$k_c = \begin{bmatrix} 1.000 & 1.630 & 0.040 & -0.002 \end{bmatrix}$$  \hspace{1cm} (4.8b)

$$c_c = 0.600$$  \hspace{1cm} (4.8c)

$$f_{spf}' = \begin{bmatrix} k_t' \\ v_t'\% \\ l/d \\ f_{tf}' \end{bmatrix} + c_t$$  \hspace{1cm} (4.9a)

$$k_t = \begin{bmatrix} 0.060 & 0.790 & 0.020 & -0.001 \end{bmatrix}$$  \hspace{1cm} (4.9b)

$$c_t = 0.920$$  \hspace{1cm} (4.9c)
4.3.4 Performance of the Proposed ANN Compressive Strength and ANN Splitting Tensile Strength Models

Equations 4.8a and 4.9a represented proportional factors $k_c$ and $k_t$ of the proposed ANN compressive strength model and ANN splitting tensile strength model, respectively. These factors were used to investigate the contribution of each input parameters, i.e., $f_{c'}^c$, $v_f$, $l/d$, and $f_{tf}$ on the compressive strength and splitting tensile strength of SFRC. In general, the volume content of steel fibre has a greater influence than the aspect ratio of the fibres on the compressive strength and splitting tensile strength of SFRC. However, the contribution of the nominal yield tensile stress of steel fibre has negligible effect on the compressive strength and splitting tensile strength of SFRC.

The performance of the proposed ANN compressive strength model of SFRC is shown in Figures 4.10. In general, it can be seen from the Figure 4.10 that the proposed ANN compressive strength model of SFRC can accurately predict the experimental results. It was found that the coloration coefficient ($R^2$) between the predicted results and experimental results of the compressive strength of SFRC was 0.96.
The performance of the proposed ANN splitting tensile strength model of SFRC is shown in Figures 4.11. In general, it can be seen from the Figure 4.11 that the proposed ANN splitting tensile model of SFRC can accurately predict the experimental results. It was found that the coloration coefficient ($R^2$) between the predicted results and experimental results of the splitting tensile strength of SFRC was 0.86.
4.3.5 Verification of the Proposed ANN Compressive Strength and ANN Splitting Tensile Strength Models

In this study, a statistical indicator, i.e., Mean Average Error (MAE) was used to verify the proposed ANN compressive strength model and ANN splitting tensile strength model of SFRC. The prediction results of the proposed model are compared with the experimental results. The equation of MAE is presented as follows:

$$MAE = \frac{1}{N} \sum_{i=1}^{N} \left| \frac{pre_i - exp_i}{exp_i} \right|$$

(4.10)

where $pre_i$ refers to the prediction results, $exp_i$ refers to the experimental results, and $N$ refers to the total number of the experimental databases (102 databases). Figure 4.12
presents the percentage of error versus numbers of the prediction compressive strength data of SFRC. It can be seen from the Figure 4.12 that the most of the prediction ANN compressive strength data have an error less than 10%. Figure 4.13 shows the percentage of error versus numbers of the prediction splitting tensile strength data of SFRC. It can be seen from Figure 4.13 that the majority of the prediction splitting tensile strength data have an error less than 20%.

Figure 4.12. Percentage of error versus numbers of the prediction ANN compressive strength data of SFRC.
Figure 4.13. Percentage of error versus numbers of the prediction ANN splitting tensile strength data of SFRC.

4.4 Compressive Strength of SFR-HSC Square Columns

In Reinforced Concrete (RC) columns, the concrete core is restrained from lateral expansion by transverse reinforcement in the form of closed tie or helix and longitudinal bars. The increasing lateral strain on the transverse reinforcement results in increasing lateral confining pressure applied to the concrete core as explained in Chapter 3. Consequently, the strength and ductility of the concrete columns are improved. The expression for the effective lateral confinement pressure at peak concrete stress that applies to the nominal concrete core due to the action of the lateral reinforcement can be determined using Equation 3.7.
The inclusion of steel fibre into RC columns can also provide additional confinement pressure to the concrete core, which delays the early cover spalling due to improvement in tensile resistance at cover-core interface as explained in Chapter 3. Subsequently, substantial improvement in the strength and ductility of the concrete columns was investigated (Paultré et al. 2010). The confinement pressure provided by the steel fibre to the concrete core can be calculated using Equation 3.10.

The experimental investigation databases of SFR-HSC square columns tested under concentric axial load were gathered from the available research studies of Lima and Giongo (2004), Sharma et al. (2007b), Paultré et al. (2010). Only a few studies have proposed an analytical model to predict the compressive strength (Ganesan and Muthy 1990, Paultré et al. 2010). The compressive strength model proposed by Ganesan and Muthy (1990) did not consider the influence of steel fibre on the compressive strength of the NSC columns. However, Paultré et al. (2010) considered the effect of steel fibre on confinement of the concrete columns to predict the compressive strength of the 15 SFR-HSC square columns. Paultré et al. (2010) used the expression suggested by Cusson and Paultré (1995) to determine the compressive strength of the SFR-HSC square columns as a function of effective confinement index at the peak stress of the concrete columns, \( I'_c \) as shown in Equation 3.14. Paultré et al. (2010) also reported that the effective confinement index at peak stress of the SFR-HSC square columns should be calculated as functions of confinement pressure due to lateral reinforcement and steel fibre as shown in Table 3.2.

From the available research studies, it was observed that quite a limited number of research studies are available to predict the compressive strength of the SFR-HSC
square columns. Thus, it is essential to develop a new compressive strength model that accurately covers a wider range of the experimental results of the SFR-HSC square columns. In this study, ANN analysis is used to predict the compressive strength of SFR-HSC square concrete as a function of the compressive strength of concrete, confinement pressure provided by the lateral reinforcement and confinement pressure provided by the steel fibres.

4.4.1 Experimental Databases of SFR-HSC Square Columns
The experimental investigation databases of 69 SFR-HSC square columns were compiled and analysed. The experimental databases of SFR-HSC square columns included only RC columns tested under concentric axial load. A number of research studies have been excluded due to incomplete information in their studies. In this study, three input parameters i.e., compressive strength of concrete ($f'_c$), effective confinement pressure provided by the lateral reinforcement ($f_{le}$) and confinement pressure provided by the steel fibres ($f_b$), and one target (experimental output) parameter i.e. compressive strength ($f'_{cc}$) of SFR-HSC square columns were used to predict ANN $f'_{cc}$ model. Equations 3.7 and Equation 3.10 were used to calculate $f_{le}$ and $f_b$, respectively. From the previous experimental results, the values of $f'_{cc}$ of the square columns was calculated by subtracting the load carrying of the longitudinal bars from the total loads divided by concrete core area. Table 4.2 presents the experimental investigation databases of SFR-HSC square columns. It was found from Table 4.2 that the range of the $f'_c$ ranged between 61 MPa and 101 MPa. The volume content of steel fibre ranged between 0.25% and 2%. The aspect ratio of steel fibre ranged between 20 and 80. Two types of steel fibres, i.e., hooked end steel fibre and crimped steel fibre, were involved in the experimental investigation databases. In addition, the longitudinal reinforcement ratio
ranged between 2.01% and 4.02%. The yield tensile stress of the longitudinal reinforcement ranged between 395 MPa and 597 MPa. The lateral reinforcement ranged between 0.32% and 5.6%. The yield tensile stress of the lateral reinforcement ranged between 392 MPa and 856 MPa. Two types of square cross-sections columns of 150 mm and 235 mm were used in this study. Moreover, the \( f'_{cc} \) ranged between 45 MPa and 155 MPa.
Table 4.2. Experimental investigation databases of SFR-NSC and SFR-HSC square columns.

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4.4.2 Design ANN Analysis for Compressive Strength of SFR-HSC Square Columns

In this study three input parameters ($f'_c$, $f_i$ and $f_b$) and one target parameter ($f_{cc}'$) were used to design the ANN analysis. After designing, the neural network is configured. A Feed-forward back propagation type of network was used. A Levenberg-Marquardt (LM) algorithm type of back propagation, was used. The Trainlm function of training, the Learngdm adoption learning function and the MSE performance function were used. Two types of transfer functions, i.e., Purelin and Tansig functions were used to develop and train the neural network. All input and target data were randomly divided into training (70%), testing (15%), and validation (15%). The network architecture of the proposed ANN compressive strength models of and SFR-HSC square columns is presented in Figure 4.14.

Figure 4.14. The network architecture of the proposed ANN compressive strength models of SFR-HSC square columns.
4.4.3 Proposed ANN Compressive Strength Model of SFR-HSC Square Columns

The Purelin function was developed to predict the $f_{cc}'$ of square columns. After training, the input weight matrix ($IW$), the output weight matrix ($LW$) and bias matrices $b_1$ and $b_2$ were obtained. Equation 4.1 was used to modify the Purelin strength model as given below:

$$y = LW \times IW \times X_n + LW \times b_1 + b_2$$

(4.11)

$$y = \sum_{n=1}^{3} W \times X_n + LW \times b_1 + b_2$$

(4.12)

Based on the trained ANN, Equations 4.11 and 4.12 can be used to develop the ANN Purelin compressive strength model of SFR-HSC square columns and is expressed as follows:

$$f_{cc}' = [k_{cc}] [f_{c}] - c_{cc}$$

(4.13a)

$$k_{cc} = [1.14, 0.89, 2.09]$$

(4.13b)

$$c_{cc} = -7.8$$

(4.13c)

where the values of 2.09 is dependent on the confinement pressure provided by steel fibres.
The ANN Tansig compressive strength model of SFR-HSC square columns was trained using Tansig transfer function. The ANN Tansig compressive strength model cannot be simplified to the form of ANN Purelin compressive strength model. The input weight matrix \((IW)\), the output weight matrix \((LW)\) and bias matrices \(b_1\) and \(b_2\) were obtained. By substituting the \(IW\), \(LW\), \(b_1\) and \(b_2\) matrices into Equation 4.17, the compressive strength of SFR-HSC square columns can be determined, and it is expressed as follows:

\[
f'_{c \nu} = \tanh \left[ LW^T \left[ \tanh \left( \sum_{n=1}^{3} IW \times (X_n)^T + b_1 \right) \right] + b_2 \right]
\]  

(4.17)

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<td>-4.55</td>
</tr>
<tr>
<td>-1.21</td>
</tr>
<tr>
<td>1.71</td>
</tr>
<tr>
<td>2.97</td>
</tr>
<tr>
<td>1.54</td>
</tr>
<tr>
<td>-2.32</td>
</tr>
<tr>
<td>1.91</td>
</tr>
<tr>
<td>1.73</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>(b_2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
</tr>
<tr>
<td>1.32</td>
</tr>
<tr>
<td>1.26</td>
</tr>
<tr>
<td>-0.39</td>
</tr>
<tr>
<td>2.05</td>
</tr>
<tr>
<td>-0.76</td>
</tr>
<tr>
<td>2.64</td>
</tr>
<tr>
<td>-1.43</td>
</tr>
<tr>
<td>-1.18</td>
</tr>
<tr>
<td>-1.55</td>
</tr>
<tr>
<td>-2.38</td>
</tr>
<tr>
<td>1.41</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>(-2.69)</th>
</tr>
</thead>
</table>

\(b_2 = 0.24\)

### 4.4.4 Performance of the Proposed ANN Purelin and Tansig Compressive Strength Models

In ANN Purelin compressive strength model, the contribution of the input parameters, \(i.e., f'_{c}, f_t\) and \(f_b\) to the prediction of ANN output can be investigated based on the proportional factors \((k)\) as presented in Equation 4.13a. The three input parameters \((f'_{c},\)
$f_l$ and $f_b$) significantly influence the compressive strength of SFR-HSC square columns. It was observed that the contribution of the input $f_b$ to the $f'_{ec}$ is greater than that of the $f_l$. Consequently, increasing the amount of the steel fibre has significantly influenced the strength of square concrete columns compared with an increasing amount of lateral reinforcement. In addition, a partial replacement of lateral reinforcement with steel fibres in RC columns can be recommended to reduce the cost of the structural members. The same outcome was found by Ganesan and Murthy (1990).

The performance of the developed ANN Purelin and ANN Tansig compressive strength models are shown in Figure 4.15. It can be seen from Figure 4.15 that the developed ANN Purelin and ANN Tansig compressive strength models for a wider range of the experimental databases of SFR-HSC square columns provided prediction results that fit the experimental results very well. It was found the correlation factor ($R^2$) between the prediction results obtained from ANN Purelin compressive strength model and the experimental results was 0.72. Whereas, the correlation factor ($R^2$) between the prediction results obtained from the ANN Tansig compressive strength model and experimental results was 0.93.
Figure 4.15. Comparison between the prediction and experimental results of the proposed (a) ANN Purelin and compressive strength model, and (b) ANN Tansig compressive strength model.
4.4.5 Verification of the Proposed ANN Purelin and ANN Tansig Compressive Strength Models

The statistical indicator of MAE (Equation 4.10) was used to verify the performance of the ANN Purelin and ANN Tansig compressive strength models. The developed ANN Purelin compressive strength model for a wider range of data point shows slightly higher error than the ANN Tansig compressive strength model. The majority of predict compressive strength results using ANN Purelin compressive strength model have an error less than 20% as shown in Figure 4.16. However, the most of the predict compressive strength results using ANN Tansig compressive strength model have error lower than 10% as shown in Figure 4.17.

![Figure 4.16. Percentage of error versus numbers of the prediction Purelin compressive strength data of SFRC square columns.](image-url)
4.5. Summary

This chapter presents two sets of experimental investigation databases. The first set was experimental investigation databases of SFRC which were used to predict the ANN compressive strength model and ANN splitting tensile strength model of SFRC. The second set was experimental investigation databases of SFR-HSC square columns which were used to propose ANN Purelin compressive strength model and ANN Tansig compressive strength model of SFR-HSC square columns. The proposed models cover a wide range of experimental investigation databases. It was observed that the ANN analysis can be accurately used to predict strength models.
The following chapters present the details of the pilot study, experimental program and experimental results and discussion of HSC and SFR-HSC circular specimens tested under different load condition (concentric load, 25 mm and 50 mm eccentric loads and four-point load). Furthermore, the analytical results of the experimental program are presented and discussed.
5: PILOT STUDY OF STEEL FIBRE REINFORCED HIGH STRENGTH CONCRETE

5.1 General

In this chapter, the details of the pilot study of Steel Fibre Reinforced High Strength Concrete (SFR-HSC) are presented. The pilot study was conducted to investigate the influence of the main parameters such as fibre type (micro steel fibres, macro steel fibres, and hybrid steel fibres), volume content and aspect ratio of steel fibres on the behaviour of SFR-HSC was examined. The effect of fibres type, volume content and aspect ratio (length/diameter) of steel fibres on the workability, compressive strength and splitting tensile strength of SFR-HSC. The pilot study was conducted at the structural laboratory of the School of Civil, Mining and Environmental Engineering at the University of Wollongong, Australia. The following sections of this chapter present details of the pilot study.

5.2 Materials

In order to produce High Strength Concrete (HSC) with and without steel fibres mixes, the following materials were used: Ordinary Portland Cement (OPC) Type I, fly ash as a partial replacement of 7% of cement, fine aggregate with a maximum size of 4.75 mm, coarse aggregate with a maximum aggregate size of 10 mm, superplasticizer and three types of steel fibres. The first type of the fibre was micro steel fibre (copper-coated micro steel fibre), which was supplied by Ganzhou Daye Metallic Fibres (GDMF, 2014). The second type of the fibre was macro steel fibre (deformed macro steel fibre), which was provided by Fibercon Co. Ltd (Fibercon, 2014). The third type of the fibres
was hybrid steel fibres, which is composed of a combination of micro steel fibres and macro steel fibres. Table 5.1 shows the nominal properties of steel fibres that were provided by the manufacturers. These properties conform with A820M-11 (ASTM 2011). The nominal aspect ratio (length/diameter) of the micro steel fibres and macro steel fibres were 30 and 33, respectively. Figure 5.1 shows the shape of steel fibres.

Table 5.1. Nominal properties of steel fibres provided by manufacturers.

<table>
<thead>
<tr>
<th>Type of steel fibres</th>
<th>Length ($l$) (mm)</th>
<th>Diameter ($d$) (mm)</th>
<th>Aspect ratio ($l/d$)</th>
<th>Tensile strength (MPa)</th>
<th>Density of fibre (kg/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Micro steel fibres (GDMF, 2014)</td>
<td>6±1</td>
<td>0.2±0.05</td>
<td>30</td>
<td>&gt;2600</td>
<td>7900</td>
</tr>
<tr>
<td>Macro steel fibres (Fibercon, 2014)</td>
<td>18</td>
<td>0.55</td>
<td>33</td>
<td>800</td>
<td>7865</td>
</tr>
</tbody>
</table>

Figure 5.1. The shape of steel fibres.

5.3 Mix Proportions

A total of 40 cylindrical specimens were prepared from ten HSC mixes. Four cylinders of 100 mm x 200 mm from each mix were cast and tested. Mix R included plain HSC without steel fibres. Mixes MI-1, MI-2, and MI-3 included 2%, 3%, and 4% by volume of micro steel fibres, respectively. Mixes MA-1, MA-2, and MA-3 included 1%, 2%, and 3% by volume of macro steel fibres, respectively. Mixes HY-1, HY-2, and HY-3
included 1.5% (1% micro + 0.5% macro), 2.5% (1.5% micro + 1% macro), and 3.5% (2% micro + 1.5% macro) by volume of hybrid steel fibres, respectively. The mix proportions for 1 m$^3$ of the HSC with and without steel fibres are presented in Table 5.2. The amount of superplasticizer (high range water reducers) varied from 1.5% to 2% of the weight of binder material (Cement + Fly ash) to maintain workability of the mixes.

5.4 Mixing and Curing Processes

A mixer with a capacity of 0.2 m$^3$ was used to produce the HSC. First, the fine and coarse aggregates were added to the mixer and mixed for 3 minutes. Then the cement and fly ash were added and mixed for an additional 3 minutes. The steel fibres were added to the concrete mixture in dry phase, and the mixing process continued for about 3 minutes until a homogenous mixture was obtained. The amount of steel fibres (micro, macro and hybrid) was added by volume into mix design as a partial replacement of aggregate. Finally, water was added to the mixture along with the superplasticizer and mixed for an additional 3 minutes to obtain a homogeneous wet mixture. The same process was followed to produce all other concrete mixtures. The workability of the fresh concrete mixtures was conducted using slump test according to AS 1012.3.1-14 (Australian Standards 2014) as shown in Figure 5.2.

The fresh concrete mixtures were poured into cylinder moulds and vibrated to reduce air bubbles. After 24 hours of placing the concrete, the specimens were demoulded and cured in the water tank at room temperature. After 28 days of casting the mixes, the compression test and tension test (splitting tensile test) were performed according to AS 1012.9-14 (Australian Standards 2014).
Table 5.2. Mix proportions for 1 m$^3$ of the HSC with and without steel fibres.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Water (kg/m$^3$)</th>
<th>Cement (kg/m$^3$)</th>
<th>Fine aggregate (kg/m$^3$)</th>
<th>Coarse aggregate (kg/m$^3$)</th>
<th>Fly ash (kg/m$^3$)</th>
<th>Volume content of Micro steel fibres (%)</th>
<th>Macro steel fibres (%)</th>
<th>Hybrid steel Fibres (%)</th>
<th>Unite weight of mix (kg/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>190</td>
<td>505</td>
<td>726</td>
<td>1059</td>
<td>35</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2486</td>
</tr>
<tr>
<td>MI-1</td>
<td>151</td>
<td>504</td>
<td>704</td>
<td>1056</td>
<td>35</td>
<td>2</td>
<td>-</td>
<td>-</td>
<td>2499</td>
</tr>
<tr>
<td>MI-2</td>
<td>151</td>
<td>504</td>
<td>702</td>
<td>1054</td>
<td>35</td>
<td>3</td>
<td>-</td>
<td>-</td>
<td>2513</td>
</tr>
<tr>
<td>MI-3</td>
<td>151</td>
<td>504</td>
<td>701</td>
<td>1051</td>
<td>35</td>
<td>4</td>
<td>-</td>
<td>-</td>
<td>2526</td>
</tr>
<tr>
<td>MA-1</td>
<td>151</td>
<td>504</td>
<td>706</td>
<td>1059</td>
<td>35</td>
<td>-</td>
<td>1</td>
<td>-</td>
<td>2473</td>
</tr>
<tr>
<td>MA-2</td>
<td>151</td>
<td>504</td>
<td>704</td>
<td>1056</td>
<td>35</td>
<td>-</td>
<td>2</td>
<td>-</td>
<td>2486</td>
</tr>
<tr>
<td>MA-3</td>
<td>151</td>
<td>504</td>
<td>702</td>
<td>1054</td>
<td>35</td>
<td>-</td>
<td>3</td>
<td>-</td>
<td>2499</td>
</tr>
<tr>
<td>HY-1</td>
<td>151</td>
<td>504</td>
<td>705</td>
<td>1058</td>
<td>35</td>
<td>-</td>
<td>-</td>
<td>1.5 (1% micro + 0.5% macro)</td>
<td>2480</td>
</tr>
<tr>
<td>HY-2</td>
<td>151</td>
<td>504</td>
<td>703</td>
<td>1055</td>
<td>35</td>
<td>-</td>
<td>-</td>
<td>2.5 (1.5% micro + 1% macro)</td>
<td>2493</td>
</tr>
<tr>
<td>HY-3</td>
<td>151</td>
<td>504</td>
<td>702</td>
<td>1052</td>
<td>35</td>
<td>-</td>
<td>-</td>
<td>3.5 (2% micro + 1.5% macro)</td>
<td>2506</td>
</tr>
</tbody>
</table>
5.5 Results

5.5.1 Slump Test

The results of the slump test of fresh HSC with and without steel fibres are presented in Table 5.3. Figure 5.3 shows the results of the slump test of the fresh SFR-HSC. It can be seen from Figure 5.3 that the slump values of SFR-HSC decreased with an increase of volume content of steel fibres. The slump of Mixes MI-1, MI-2, and MI-3 decreased by 38%, 68%, and 94%, respectively, compared to Mix R. The slump decreased by 56%, 75%, and 94% for Mixes MA-1, MA-2, and MA-3, respectively, compared to Mix R. Moreover, the slump of Mixes HY-1, HY-2, and HY-3 decreased by 63%, 80%, and 100%, respectively, compared to Mix R.

It was found that the slump of SFR-HSC decreased with an increase of aspect ratio (length/diameter) of steel fibres. For the same volume content of 2% and 3% of micro steel fibres and macro steel fibres, the slump of SFR-HSC reduced with an increase of aspect ratio of steel fibres. The slump of Mixes MI-1 and MA-2 reduced by 38% and 75%, respectively, compared to Mix R. In addition, the slump decreased by 69% and
94% for Mixes MI-2 and MA-3, respectively, compared to Mix R. Thus, the reasonable reduction in the slump was found with Mixes MI-2, MA-2, and HY-2.

Table 5.3. Results of the slump test of fresh HSC with and without steel fibres.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Slump (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>80</td>
</tr>
<tr>
<td>MI-1</td>
<td>50</td>
</tr>
<tr>
<td>MI-2</td>
<td>25</td>
</tr>
<tr>
<td>MI-3</td>
<td>5</td>
</tr>
<tr>
<td>MA-1</td>
<td>35</td>
</tr>
<tr>
<td>MA-2</td>
<td>20</td>
</tr>
<tr>
<td>MA-3</td>
<td>5</td>
</tr>
<tr>
<td>HY-1</td>
<td>30</td>
</tr>
<tr>
<td>HY-2</td>
<td>16</td>
</tr>
<tr>
<td>HY-3</td>
<td>0</td>
</tr>
</tbody>
</table>

Figure 5.3. Results of the slump test of SFR-HSC.

5.5.2 Failure Modes of the Mixes

The typical failure modes of the mixes tested under compression are shown in Figure 5.4. It was observed that Mix R failed in a brittle manner as shown in Figure 5.4 (a).
However, with the inclusion of steel fibres into HSC, the failure modes became a combination of compression failure and bugling of the mixes in the lateral direction. The effect of the volume content of steel fibres on the failure mode of concrete cylinders was quite insignificant compared to type of steel fibres. Thus, type of steel fibres (micro, macro and hybrid steel fibres) were considered in determining failure modes of concrete cylinders. Mixes MI failed in compression, and the diagonal shear plane was observed for all Mixes MI as shown in Figure 5.4 (b). Mixes MA failed in compression, and the lateral bugling was observed for all Mixes MA as shown in Figure 5.4 (c). The failure mode of all Mixes HY was due to bugling of the mixes in the lateral direction as shown in Figure 5.4 (d). A better performance of the concrete reinforced with different types of steel fibres was based on the types of steel fibres and the maximum axial compression load carried by the cylinders. It was observed from the failure modes of the mixes tested under compression that Mixes HY presented a better performance compared to Mixes MI and MA.

The typical failure modes of the mixes tested under tension are shown in Figure 5.5. It was found that Mix R split into two parts as shown in Figure 5.5 (a). However, the mixes reinforced with steel fibres did not split under the tension load. The failure modes of Mixes MI and MA were due to the propagation of large cracks parallel to the loading direction as shown in Figure 5.5 (b) and (c). The failure mode of Mixes HY was due to small cracks parallel to the load direction as shown in Figure (d). Thus, it was observed from the failure modes of the mixes tested under tension load that Mixes HY presented a better performance compared to Mixes MI and MA.
Figure 5.4. Typical failure modes of the mixes: (a) R, (b) MI, (c) MA and (d) HY tested under compression load.
Figure 5.5. Typical failure modes of the mixes: (a) R, (b) MI, (c) MA and (d) HY tested under splitting tensile.

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5.5.3 Compressive Strength

The axial compression test was performed on two cylinders of 100 mm x 200 mm for each mix. Figure 5.6 shows the influence of the volume content of micro steel fibres on the average compressive strength of SFR-HSC. The results of the compressive strength of the HSC with and without steel fibres are presented in Table 5.4. It was observed that the compressive strength of the HSC was marginally affected by the inclusion of the different types of steel fibre. It was found that the improvement in the compressive strength of Mixes MI ranged between 1% and 6%. Higher improvement in compressive strength was observed with Mix MI-2. The average compressive strength increased by 3%, 6%, and 1% for Mixes MI-1, MI-2, and MI-3, respectively, compared to Mix R.

![Graph showing the influence of the volume content of micro steel fibres on the average compressive strength of SFR-HSC.]

Figure 5.6. The influence of the volume content of micro steel fibres on the average compressive strength of SFR-HSC.
Table 5.4. Results of compressive strength of HSC with and without steel fibres.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cylinder No.</th>
<th>Compressive strength (MPa)</th>
<th>Average compressive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>1</td>
<td>66.8</td>
<td>65.0</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>63.2</td>
<td></td>
</tr>
<tr>
<td>MI-1</td>
<td>1</td>
<td>68.1</td>
<td>66.7</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>65.3</td>
<td></td>
</tr>
<tr>
<td>MI-2</td>
<td>1</td>
<td>69.5</td>
<td>68.9</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>68.3</td>
<td></td>
</tr>
<tr>
<td>MI-3</td>
<td>1</td>
<td>65.4</td>
<td>65.8</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>66.2</td>
<td></td>
</tr>
<tr>
<td>MA-1</td>
<td>1</td>
<td>64.2</td>
<td>64.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>64.8</td>
<td></td>
</tr>
<tr>
<td>MA-2</td>
<td>1</td>
<td>65.5</td>
<td>65.8</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>66.1</td>
<td></td>
</tr>
<tr>
<td>MA-3</td>
<td>1</td>
<td>65.0</td>
<td>65.1</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>65.2</td>
<td></td>
</tr>
<tr>
<td>HY-1</td>
<td>1</td>
<td>66.2</td>
<td>65.2</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>64.2</td>
<td></td>
</tr>
<tr>
<td>HY-2</td>
<td>1</td>
<td>70.1</td>
<td>68.4</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>66.7</td>
<td></td>
</tr>
<tr>
<td>HY-3</td>
<td>1</td>
<td>71.0</td>
<td>68.0</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>63.1</td>
<td></td>
</tr>
</tbody>
</table>

Figure 5.7 shows the effect of the volume of macro steel fibres on the average compressive strength of SFR-HSC. It was observed that the compressive strength of the HSC was marginally affected by the addition of macro steel fibres. Mix MA-2 showed slightly better compressive strength than Mixes R, MA-1, and MA-3. The average compressive strength of Mix MA-1 decreased by 1% compared to Mix R. However, the average compressive strength increased by 1% and 0.2% for Mixes MA-2 and MA-3, respectively, compared to Mix R.
Figure 5.7. The influence of the volume content of macro steel fibres on the average compressive strength of SFR-HSC.

Figure 5.8 presents the influence of volume content of hybrid steel fibres on the average compressive strength of SFR-HSC. It was found that the improvement in the compressive strength of Mixes HY ranged between 0.3% and 5%. The higher improvement in compressive strength was observed with Mixes HY-2 and HY-3 compared to Mixes HY-1 and R. The average compressive strength of Mixes HY-2 and HY-3 increased by 5% compared to Mix R. However, the average compressive strength increased by 0.3% for Mix HY-1 compared to Mix R.
The influence of the volume content of hybrid steel fibres on the average compressive strength of SFR-HSC was studied by considering volume contents of 2% and 3%. Figure 5.9 presents the influence of the aspect ratio of steel fibres on the average compressive strength of SFR-HSC. In general, it was found that the compressive strength of SFR-HSC decreased with an increase of the aspect ratio of steel fibres. It was found that the average compressive strength increased by 3% and 1% for Mixes MI-1 and MA-2, respectively, compared to Mix R. However, the average compressive strength increased by 6% and 0.2% for Mixes MI-2 and MA-3, respectively, compared to Mix R. It was observed that using a low aspect ratio of steel fibres including micro steel fibres leads to improving the compressive strength of SFR-HSC more than using a high aspect ratio of steel fibres including macro steel fibres of
the same volume content. Furthermore, it was found that with the use of 2% by volume of the fibres, an increase in the aspect ratio of the fibres resulted in reducing the average compressive strength by 2% compared to Mix R. However, with the use of 3% by volume of the fibres, an increase in aspect ratio of the fibres resulted in decreasing the average compressive strength by 6% compared to Mix R.

Figure 5.9. The effect of the aspect ratio of steel fibres on the average compressive strength of SFR-HSC.

5.5.4 Splitting Tensile Strength

The tension test (splitting tensile test) was performed on two cylinders of 100 mm x 200 mm for each mix. Figure 5.10 presents the influence of the volume content of micro steel fibres on the average splitting tensile strength of SFR-HSC. The results of the splitting tensile strength of the HSC with and without steel fibres are shown in Table 5.5. It was found that the splitting tensile strength of the HSC significantly improved
with the inclusion of steel fibres. The splitting tensile strength increased with the increase in the volume content of steel fibres. The improvement in the splitting tensile strength of Mixes MI ranged between 56% and 77%. A higher improvement in splitting tensile strength was observed with Mix MI-3. The average splitting tensile strength increased by 56%, 74%, and 77% for Mixes MI-1, MI-2, and MI-3, respectively, compared to Mix R.

Figure 5.10. The influence of the volume content of micro steel fibres on the average splitting tensile strength of SFR-HSC.
Table 5.5. Results of the splitting tensile strength of HSC with and without steel fibres.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cylinder No.</th>
<th>Split tensile strength (MPa)</th>
<th>Average split tensile strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>1</td>
<td>3.5</td>
<td>3.9</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>4.3</td>
<td></td>
</tr>
<tr>
<td>MI-1</td>
<td>1</td>
<td>6.0</td>
<td>6.1</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>6.1</td>
<td></td>
</tr>
<tr>
<td>MI-2</td>
<td>1</td>
<td>7.1</td>
<td>6.8</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>6.5</td>
<td></td>
</tr>
<tr>
<td>MI-3</td>
<td>1</td>
<td>6.6</td>
<td>6.9</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>7.2</td>
<td></td>
</tr>
<tr>
<td>MA-1</td>
<td>1</td>
<td>5.7</td>
<td>6.2</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>6.6</td>
<td></td>
</tr>
<tr>
<td>MA-2</td>
<td>1</td>
<td>6.5</td>
<td>7.0</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>7.4</td>
<td></td>
</tr>
<tr>
<td>MA-3</td>
<td>1</td>
<td>7.0</td>
<td>7.0</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>7.0</td>
<td></td>
</tr>
<tr>
<td>HY-1</td>
<td>1</td>
<td>6.3</td>
<td>6.3</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>6.2</td>
<td></td>
</tr>
<tr>
<td>HY-2</td>
<td>1</td>
<td>7.3</td>
<td>7.2</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>7.1</td>
<td></td>
</tr>
<tr>
<td>HY-3</td>
<td>1</td>
<td>8.0</td>
<td>7.8</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>7.6</td>
<td></td>
</tr>
</tbody>
</table>

Figure 5.11 presents the influence of the volume content of macro steel fibres on the average splitting tensile strength of SFR-HSC. The improvement in the splitting tensile strength of Mixes MA ranged between 59% and 79%. A higher improvement in splitting tensile strength was observed with Mixes MA-2 and MA-3. The average splitting tensile strength of Mixes MA-2 and MA-3 increased by 79% compared to Mix R. However, the average splitting tensile strength increased by 59% for Mix MA-1 compared to Mix R.
Figure 5.11. The influence of the volume content of steel fibres on the average splitting tensile strength of SFR-HSC.

Figure 5.12 presents the influence of volume content of hybrid steel fibres on the average splitting tensile strength of SFR-HSC. It was observed that the splitting tensile strength of HSC significantly improved with the inclusion of hybrid steel fibres. The improvement in the splitting tensile strength of Mixes HY ranged between 62% and 100%. A higher improvement in splitting tensile strength was observed with Mix HY-3 compared to Mix R. The average splitting tensile strength increased by 62%, 85%, and 100% of Mixes HY-1, HY-2, and HY-3, respectively, compared to Mix R.
The influence of the volume content of hybrid steel fibres on the average splitting tensile strength of SFR-HSC was studied by considering the volume contents of 2% and 3%. Figure 5.13 presents the influence of the aspect ratio of steel fibres on the average splitting tensile strength of SFR-HSC. In general, it was found that the splitting tensile strength of SFR-HSC increased with an increase of the aspect ratio of steel fibres. It was observed that the average splitting tensile strength increased by 55% and 78% for Mixes MI-1 and MA-2, respectively, compared to Mix R. Whereas, the average splitting tensile strength increased by 74% and 79% for Mixes MI-2 and MA-3, respectively, compared to Mix R. It was found that with the use of 2% by volume of fibres, an increase in the aspect ratio of fibres leads to increasing the average splitting tensile strength by 23% compared to Mix R. However, with the use of 3% by volume of fibres,
an increase in aspect ratio of the fibres results in increasing the average splitting tensile strength by 5% compared to Mix R.

![Graph showing the effect of aspect ratio on splitting tensile strength](image)

Figure 5.13. The effect of the aspect ratio of steel fibres on the average splitting tensile strength of SFR-HSC.

### 5.6 Summary

In this chapter a pilot study of SFR-HSC was presented. The influence of different type (micro, macro, and hybrid), volume content, and aspect ratio of steel fibres on the workability, compressive strength and splitting tensile strength of SFR-HSC was observed. The results of the pilot study were presented. The inclusion of 3%, 2%, and 2.5% by volume of micro, macro, and hybrid steel fibres, respectively, into the HSC produced a reasonable improvement in strength of the HSC with a low reduction in the workability of the fresh HSC mixture. The next chapter describes the behaviour of the steel reinforced SFR-HSC columns under different loading conditions (concentric load, 25 mm, 50 mm eccentric load and four-point bending).
6: EXPERIMENTAL PROGRAM

6.1 General

This chapter presents details of the experimental program that was conducted to investigate the behaviour of circular Steel Fibre Reinforced High Strength Concrete (SFR-HSC) specimens under different loading conditions. In this chapter, the details of the materials used in this experimental program are briefly described. The description of the experimental program and fabrication of the tested specimens including design, casting, curing and instrumentation of the specimens are illustrated. Also, the testing procedure for the specimens tested under different loading conditions including concentric axial load, eccentric axial load, and four-point bending are presented. The experimental work has been conducted at the structural engineering laboratory of the School of Civil, Mining, and Environmental Engineering at the University of Wollongong, Australia. The details of the experimental program are described in the following sections.

6.2 Materials

The materials used in this experimental work are High Strength Concrete (HSC), longitudinal and helical steel reinforcement, and different types of steel fibres including micro steel fibre, macro steel fibre, and hybrid steel fibre.

6.2.1 Concrete

Ready-mix HSC was provided by a local concrete supplier to cast all specimens. An average concrete compressive strength of 60 MPa at 28 days was designed to be used.
Table 6.1 presents the mix proportion for 1 m$^3$ of the HSC provided by the local supplier (Boral 2014). The maximum size of the coarse aggregate was 10 mm.

Table 6.1. Mix proportion for 1 m$^3$ of the HSC provided by the local supplier (Boral 2014).

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cement (kg/m$^3$)</th>
<th>Water (kg/m$^3$)</th>
<th>Fly ash (kg/m$^3$)</th>
<th>Fine aggregate (kg/m$^3$)</th>
<th>Coarse aggregate (kg/m$^3$)</th>
<th>Superplasticizer (ml/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSC</td>
<td>461</td>
<td>141</td>
<td>29</td>
<td>850</td>
<td>1175</td>
<td>1400</td>
</tr>
</tbody>
</table>

### 6.2.2 Steel Fibres

Three types of steel fibres: micro steel fibre, macro steel fibre and hybrid steel fibres, as shown in Figure 5.1, are used in the experimental program. Based on the results obtained from Chapter 5, the volume content of 3% of micro steel fibres, 2% of macro steel fibres, and 2.5% of hybrid steel fibres were used in this experimental program.

### 6.2.3 Steel Reinforcement

Two diameters of steel reinforcement were used in this study. The longitudinal deformed steel reinforcement of 12 mm (N12) diameter with a nominal yield tensile strength of 500 MPa and the helix plane steel reinforcement of 10 mm (R10) diameter with a nominal tensile yield strength of 250 MPa were used. Two samples of N12 and two samples of R10 steel reinforcement were randomly selected and tested according to AS 1391-07 (Australian Standards 2007) to determine the mechanical properties of the steel reinforcement. The total length of the samples was 500 mm including 80 mm gripping length at both ends of the bars and 340 mm clear test length. Figure 6.1 shows the tensile test of the steel reinforcement. Table 6.2 presents a summary of the results of
tensile yield strength and corresponding strain of N12 and R10 steel reinforcement. Figure 6.2 shows the tensile stress-strain curves of the steel reinforcement.

Figure 6.1. Tensile test of the steel reinforcement.

Table 6.2. Results of tension test of steel reinforcement bars.

<table>
<thead>
<tr>
<th>Bar designation</th>
<th>Sample no.</th>
<th>Yield tensile strength (MPa)</th>
<th>Average of yield tensile strength (MPa)</th>
<th>Strain at yield tensile stress</th>
<th>Average of strain at yield tensile stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>N12</td>
<td>1</td>
<td>501</td>
<td>483</td>
<td>0.0026</td>
<td>0.0023</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>464</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>R10</td>
<td>1</td>
<td>330</td>
<td>328</td>
<td>0.0020</td>
<td>0.0018</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>325</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 6.2. Tensile stress-strain curves of (a) N12 and (b) R10 steel reinforcement.
6.3 Description of the Experimental Program

The experimental program was divided into the preliminarily testing program and the main experimental program. The details of the experimental program are presented in the following sections.

6.3.1 Preliminarily Testing Program

The preliminarily testing program was conducted to examine the material properties of HSC and SFR-HSC. The compression test and splitting tensile test were conducted on the HSC and SFR-HSC in the preliminarily testing program. The results of the preliminary testing were used for the analytical modelling of the tested specimens which are discussed in detail in Chapter 8. The preliminary testing program was divided into four groups of nine cylinders. In each group, three cylinders of 150 mm x 300 mm were cast and tested to investigate the compressive stress-strain response of the HSC and SFR-HSC, three cylinders of 100 mm x 200 mm were cast and tested to investigate the compressive strength of the HSC and SFR-HSC, and three cylinders of 100 mm x 200 mm were cast and tested to investigate the tensile strength of the HSC and SFR-HSC. These groups were extracted based on the results of Chapter 5. The first group, HSC consisted of HSC with no steel fibres. The second group, MI consisted of HSC reinforced with 3% by volume of micro steel fibres. The third group, MA consisted of HSC reinforced with 2% by volume of macro steel fibres. The fourth group, HY consisted of HSC reinforced with 2.5% by volume of hybrid steel fibres. Table 6.3 presents test matrix of the preliminary testing program. All cylinders were cast on the same day as the main experimental program. The concrete cylinders were taken out of the moulds and placed in a water tank for curing at room temperature for 28 days. The test of the cylinders was conducted according to AS 1012.9-14 (Australian Standards
The cylinders in the preliminary testing program were labelled based on the type of steel fibres and number of the cylinder. For example, MI-1 refers to the first cylinder of HSC consisted of 3% by volume of micro steel fibre tested either under compression or splitting tensile.

**Table 6.3. Test matrix of the preliminary testing program.**

<table>
<thead>
<tr>
<th>Group</th>
<th>Cylinders No.</th>
<th>Steel fibre Type</th>
<th>Volume content ($V_f$%)</th>
<th>Cylinder (mm)</th>
<th>Test at 28th days</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSC</td>
<td>HSC-1, HSC-2, HSC-3</td>
<td>None</td>
<td>None</td>
<td>150 x 300</td>
<td>Compression (for stress-strain behaviour)</td>
</tr>
<tr>
<td></td>
<td>HSC-1, HSC-2, HSC-3</td>
<td>Micro steel fibre</td>
<td>3%</td>
<td>100 x 200</td>
<td>Compression</td>
</tr>
<tr>
<td></td>
<td>HSC-1, HSC-2, HSC-3</td>
<td>Macro steel fibre</td>
<td>2%</td>
<td>100 x 200</td>
<td>Splitting tensile</td>
</tr>
<tr>
<td></td>
<td>HY-1, HY-2, HY-3</td>
<td>Hybrid fibre</td>
<td>2.5% (1.5% micro + 1% macro)</td>
<td>150 x 300</td>
<td>Compression (for stress-strain behaviour)</td>
</tr>
<tr>
<td></td>
<td>HY-1, HY-2, HY-3</td>
<td></td>
<td></td>
<td>100 x 200</td>
<td>Splitting tensile</td>
</tr>
</tbody>
</table>
6.3.2 Main Experimental Program

The main experimental program involved four steel reinforced HSC specimens and twelve SFR-HSC specimens tested under different loading conditions. In this study, the same specimen sizes were tested under different loading conditions to avoid the size effect on the experiment results. The specimens were 200 mm in diameter and 800 mm in height. These dimensions were chosen to be compatible with the conditions and capacity of the available compression testing machine in the laboratory.

The specimens were divided into four groups of four specimens. The first group, RC contained steel reinforced HSC specimens with no steel fibres, while the other three groups contained steel fibres. Steel fibres were added as a percentage by volume of the specimens. The second group, MI contained steel reinforced HSC specimens with 3% micro steel fibres. The third group, MA contained steel reinforced HSC specimens with 2% macro steel fibres. The fourth group, HY contained steel reinforced HSC specimens with 2.5% hybrid steel fibres, which was a combination of 1.5% of micro steel fibres and 1% of macro steel fibres. From each group, one specimen was tested under concentric axial compression load (0), one specimen was tested under 25 mm eccentric axial compression load (25), one specimen was tested under 50 mm eccentric axial compression load (50), and one specimen was tested as a beam under four-point bending (B). The specimens were designated based on the types of steel fibres (micro, macro and hybrid steel fibre) and the applied load conditions (concentric axial load, eccentric axial load, and four-point bending). For example, Specimen HY-25 refers to steel reinforced HSC specimens with the addition of 2.5% of hybrid steel fibres and tested under 25 mm eccentric axial load. Table 6.4 presents the test matrix of the specimens.
Table 6.4. Details of the test matrix.

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen</th>
<th>Steel fibres</th>
<th>Reinforcement</th>
<th>Test eccentricity (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Type</td>
<td>Volume content ($V_f$%)</td>
<td>Aspect ratio ($l/d$)</td>
</tr>
<tr>
<td>RC</td>
<td>RC-0</td>
<td>None</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td>RC-25</td>
<td>None</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td>RC-50</td>
<td>None</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td>RC-B</td>
<td>None</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>MI</td>
<td>MI-0</td>
<td>Micro steel fibre</td>
<td>3%</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>MI-25</td>
<td>Micro steel fibre</td>
<td>3%</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>MI-50</td>
<td>Micro steel fibre</td>
<td>3%</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>MI-B</td>
<td>Micro steel fibre</td>
<td>3%</td>
<td>30</td>
</tr>
<tr>
<td>MA</td>
<td>MA-0</td>
<td>Macro steel fibre</td>
<td>2%</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>MA-25</td>
<td>Macro steel fibre</td>
<td>2%</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>MA-50</td>
<td>Macro steel fibre</td>
<td>2%</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>MA-B</td>
<td>Macro steel fibre</td>
<td>2%</td>
<td>33</td>
</tr>
<tr>
<td>HY</td>
<td>HY-0</td>
<td>Hybrid steel fibre</td>
<td>2.5% (1.5% micro + 1% macro)</td>
<td>30 micro &amp; 33 macro</td>
</tr>
<tr>
<td></td>
<td>HY-25</td>
<td>Hybrid steel fibre</td>
<td>2.5% (1.5% micro + 1% macro)</td>
<td>30 micro &amp; 33 macro</td>
</tr>
<tr>
<td></td>
<td>HY-50</td>
<td>Hybrid steel fibre</td>
<td>2.5% (1.5% micro + 1% macro)</td>
<td>30 micro &amp; 33 macro</td>
</tr>
<tr>
<td></td>
<td>HY-B</td>
<td>Hybrid steel fibre</td>
<td>2.5% (1.5% micro + 1% macro)</td>
<td>30 micro &amp; 33 macro</td>
</tr>
</tbody>
</table>
Figure 6.3 presents the dimensions and reinforcement arrangement and configuration of the tested specimens. All specimens were reinforced longitudinally with six N12 steel bars (longitudinal reinforcement ratio = 2.06%) and lateral with R10 steel helices with a pitch of 60 mm (lateral reinforcement ratio = 3.27%). The steel reinforcement of the specimens conforms with AS 3600-09 (Australian Standards 2009).
6.4 Fabrication and Testing of the Specimens

6.4.1 Fabrication of the Specimens

Moulds for sixteen specimens made from PVC pipe with a diameter of 200 mm and a length of 800 mm were used to construct all the specimens. To prevent movement of the moulds during the casting and vibrating the concrete, all moulds were vertically fixed and tied using plywood and timber. Figure 6.4 shows the PVC moulds.

![Figure 6.4. PVC moulds.](image)

The longitudinal steel reinforcements were prepared and cut into the required lengths of 760 mm in order to have 20 mm clear cover at the top and the bottom of the specimens. Illawarra Spring Pty Ltd. (Illawarra Spring, 2014) provided the lateral steel reinforcement in the form of a coil with 160 mm outer diameter at 60 mm pitch. For all the specimens, the clear cover to the face of steel helix was 20 mm. Afterwards, the steel reinforcement was tied as a cage by assembling the helical steel and the longitudinal steel bars as shown in Figure 6.5.
6.4.2 Casting and Curing the Specimens

The specimens were cast with one batch of ready HSC obtained from a local supplier. The HSC was placed into a wheelbarrow, and Group RC was cast. In order to add steel fibres into the concrete, the HSC was placed directly from the concrete chute into the pan of concrete mixer. As per the mix design proportions, different types of steel fibre were mixed in a tray and then sprinkled into the concrete. The mixer was filled to the required height for the total volume for each group of the specimens to be 0.12 m\(^3\). The mixer was switched on for 2 to 3 minutes to have uniform mixture of steel fibres with the concrete. The HSC was poured into the moulds in three stages and were vibrated at each stage to remove air pockets. The concrete was scooped into the moulds to one-third of the total height before being vibrating with an electric vibrator. The middle and top thirds were also poured in the same manner and finally the surface was finished with a wet trowel. After casting, all the specimens were cured in the moulds for seven days by covering the specimens with wet hessian, which was covered with a plastic sheet to prevent moisture loss. Afterwards, the specimens were removed from the moulds and
cured in a similar moist condition for 21 days. Figure 6.6 shows the curing process of the specimens.

![Image of cured specimens](image)

Figure 6.6. The curing process of the specimens.

### 6.4.3 Instrumentation of the Specimens

The specimens were internally and externally instrumented. For the internal instrumentation, the axial and hoop strain in the reinforcement were measured. Two strain gauges were attached at mid-height in the two opposite side of the longitudinal bars to monitor the axial strain in the longitudinal reinforcement. Also, two strain gauges were glued at mid-height in the two opposite side of the helical bars to monitor the strain in the hoop direction. The positions of the strain gauges are presented in Figure 6.7.
For the external instrumentation, two monitoring systems were used to measure the axial and lateral deformations of the specimens. For concentric axial load, two Linear Variable Differential Transducers (LVDTs) were used to measure the axial deformation of the specimens during the test. For eccentric axial load, in addition to the two LVDTs, a laser triangulation was installed at the mid-height of the specimens to measure the lateral deformation of the specimens. Also, a laser triangulation was used to determine the midspan deflection for specimens tested under four-point bending. The laser triangulation was fixed vertically at the bottom of the specimens.
6.5 Testing Procedures

The testing procedure of the HSC and SFR-HSC is presented. In addition, the testing procedures of the specimens tested under concentric axial load, 25 mm and 50 mm eccentric axial load and four-point bending are presented in the following sections.

6.5.1 Testing of the Cylinders

A total of twelve cylinders of 150 mm x 300 mm were tested to determine the compressive stress and corresponding strain of the HSC, MI, MA, and HY cylinders. Figure 6.8 presents the compression test of the cylinder. Prior to testing, the top of the cylinders was capped with high-strength plaster to ensure uniform load face. A cage having LVDT as shown in Figure 6.8 was assembled and fixed at the mid-height of the cylinder to determine the axial strain. The cylinders were tested until failure using the Denison 5000 kN compression testing machine.

A total of twelve cylinders of 100 mm x 200 mm were tested to determine the compressive strength of the HSC, MI, MA, and HY cylinders. Prior to testing, the top of the cylinders was capped with high-strength plaster to ensure uniform load face. Figure 6.9 shows a compression test of the cylinder. The cylinders were tested in the 1800 kN Avery compression testing machine.

A total of twelve cylinders of 100 mm x 200 mm were tested to determine the splitting tensile strength of the HSC, MI, MA, and HY cylinders. Figure 6.10 shows a splitting tensile test of the cylinder. The cylinders were tested in the 1800 kN Avery compression
testing machine using a typical testing jig under an increasing load until the splitting of the cylinder as shown in Figure 6.10.

Figure 6.8. Compression test of 150 mm x 300 mm cylinder.

Figure 6.9. Compression test of 100 mm x 200 mm cylinder.
6.5.2 Specimens Tested under Axial Concentric and Eccentric Load

Twelve Specimens with diameter 200 mm and height 800 mm were tested as columns under axial concentric and eccentric load. Prior to testing, the top surfaces of the column specimens were capped with high-strength plaster to ensure uniform loading faces. Both ends of the column specimens were wrapped with the two layers of 75 mm wide Carbon Fibre Reinforced Polymer (CFRP) sheet to prevent premature failure. Two circular loading heads of high-strength steel were used for testing the specimens. For concentrically loaded specimens, the two loading heads were fixed at both ends of the specimens without using loading roll as shown in Figure 6.11. However, for eccentrically loaded specimens, a pair of loading rolls was placed in the grooves at either 25 mm or 50 mm from the centreline of the loading heads as shown in Figure 6.12.
The testing of the specimens under axial compression load started with pre-loading the specimens to 100 kN under a force controlled load at a rate of 30 kN/min and then unloaded to 20 kN. Afterwards, the test resumed under a deformation controlled load at a rate of 0.3 mm/min until the rupture of the helices reinforcement. The Denison 5000 kN compression testing machine was used to test the specimens under concentric and eccentric axial load.

![Figure 6.11. Typical test setup of the specimens tested under concentric axial load.](image)
6.5.3 Specimens Tested under Four-Point Bending

Four specimens were tested as beams under four-point bending. Two steel plate rigs were used to apply the load to the beam specimens as shown in Figure 6.13. The top plate rig has a clear span of 235 mm. The bottom plate rig has a clear span of 705 mm. In order to test the specimens under four-point bending, the bottom plate rig was diagonally placed on the loading area of the testing machine. The clear span of the top and bottom plate rigs was marked on the specimens and divided into three equal parts of 235 mm lengths. Afterwards, the specimens were lifted by a forklift and placed over the bottom plate rig on the marked lines with 47.5 mm overhang at the ends. Then the top plate rig was placed over the top of the specimens on the marked lines. Figure 6.14 shows the typical test setup of the specimens tested under four-point bending.
The testing of the specimens under four-point bending started with pre-loading the specimens to 30 kN under a force controlled load at a rate of 30 kN/min and then unloaded to 10 kN. Afterwards, the test resumed under a displacement controlled load at a rate of 0.3 mm/min until the rupture of the longitudinal reinforcement. The Denison 5000 kN compression testing machine was used to test the specimens under four-point bending.

![Steel plate rigs](image)

Figure 6.13. Steel plate rigs.
6.6 Summary

This chapter presents details of the experimental program. The fabrication, casting, curing and instrumentations of the specimens were also presented. The testing procedures of the HSC and SFR-HSC cylinders tested under compression and splitting tensile tests were explained. In addition, the testing procedures of the specimens under different loading conditions (concentric load and eccentric axial load and four-point bending) were described. The next chapter presents the experimental results and discussion of test results of the specimens.
7: EXPERIMENTAL RESULTS

7.1 General

This chapter presents the experimental results of the preliminary testing program and main experimental program. The preliminary testing program consists of compression and splitting tests of HSC, MI, MA and HY cylinders. The main experimental program consists of four steel reinforced HSC specimens and twelve steel reinforced SFR-HSC specimens tested under concentric axial load, 25 mm and 50 mm eccentric axial load and four-point bending.

The results of the preliminary testing program are discussed to present the material properties including the response of compressive stress and corresponding strain, compressive strength and splitting tensile strength of HSC, MI, MA and HY cylinders. The results of the main experimental program are discussed to determine the behaviour of SFR-HSC specimens under concentric axial load, 25 mm and 50 mm eccentric axial load and four-point bending. The following sections present the experimental results in detail.

7.2 Results of Preliminary Testing Program

The preliminary testing program was conducted in order to determine the material properties of HSC, MI, MA and HY cylinders. The results of the preliminary testing program are used in the analysis modelling of the tested specimens which are discussed in Chapter 8. The preliminary testing program results of HSC, MI, MA and HY cylinders are reported in the following sections.
7.2.1 Compressive Stress-Strain Behaviour of the Cylinders

A total of twelve cylinders of 150 mm x 300 mm were cast and tested to determine the compressive stress and corresponding strain of HSC, MI, MA and HY. The typical failure modes of the cylinders tested under compression load are shown in Figure 7.1.

Figure 7.2 presents the experimental stress-strain curves of HSC, MI, MA and HY cylinders. It was found that the compressive stress of HSC slightly increased with the inclusion of micro steel fibres. However, no improvement in compressive stress was observed with the inclusion of macro steel fibres and hybrid steel fibres into HSC. Table 7.1 shows the results of the compressive stress of HSC, MI, MA and HY cylinders. It can be seen from Table 7.1 that the average maximum compressive stress of MI cylinders increased by 2% compared to HSC cylinders. However, the average maximum compressive stress of MA and HY cylinders reduced by 13% and 2%, respectively, compared to HSC cylinders. A higher improvement in the average strain corresponding to the maximum compressive stress of SFR-HSC was observed with HY cylinders. The average strain corresponding to the maximum compressive stress increased by 4% and 21% of MI and HY cylinders, respectively, compared to HSC cylinders. This could be because the delaying in propagation of the cracks at the maximum compressive stress which leads to increasing the concrete strain. Whereas, the average strain corresponding to the maximum compressive stress of MA cylinders decreased by 11% compared to HSC cylinders.
Figure 7.1. Typical failure modes of the: (a) HSC, (b) MI, (c) MA and (d) HY cylinders tested under compression load.
Figure 7.2. Experimental results of stress-strain curves of (a) HSC, (b) MI, (c) MA, and (d) HY cylinders.
Table 7.1. Results of stress-strain relationship of HSC, MI, MA and HY cylinders.

<table>
<thead>
<tr>
<th>Group</th>
<th>Cylinder no.</th>
<th>Maximum compressive stress (MPa)</th>
<th>Average of compressive stress (MPa)</th>
<th>Strain at maximum compressive stress</th>
<th>Average strain at maximum compressive stress</th>
<th>Energy absorption capacity (N.mm/mm³)</th>
<th>Average of energy absorption capacity (N.mm/mm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSC</td>
<td>HSC-1</td>
<td>58.70</td>
<td>60.50</td>
<td>0.0026</td>
<td>0.0027</td>
<td>0.10</td>
<td>0.106</td>
</tr>
<tr>
<td></td>
<td>HSC-2</td>
<td>61.55</td>
<td></td>
<td>0.0029</td>
<td></td>
<td>0.11</td>
<td></td>
</tr>
<tr>
<td></td>
<td>HSC-3</td>
<td>61.33</td>
<td></td>
<td>0.0028</td>
<td></td>
<td>0.11</td>
<td></td>
</tr>
<tr>
<td>MI</td>
<td>MI-1</td>
<td>62.57</td>
<td>61.82</td>
<td>0.003</td>
<td>0.0028</td>
<td>0.36</td>
<td>0.323</td>
</tr>
<tr>
<td></td>
<td>MI-2</td>
<td>59.70</td>
<td></td>
<td>0.0029</td>
<td></td>
<td>0.30</td>
<td></td>
</tr>
<tr>
<td></td>
<td>MI-3</td>
<td>63.20</td>
<td></td>
<td>0.0027</td>
<td></td>
<td>0.31</td>
<td></td>
</tr>
<tr>
<td>MA</td>
<td>MA-1</td>
<td>53.23</td>
<td>52.58</td>
<td>0.0022</td>
<td>0.0024</td>
<td>0.20</td>
<td>0.222</td>
</tr>
<tr>
<td></td>
<td>MA-2</td>
<td>54.02</td>
<td></td>
<td>0.0025</td>
<td></td>
<td>0.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>MA-3</td>
<td>50.51</td>
<td></td>
<td>0.0026</td>
<td></td>
<td>0.21</td>
<td></td>
</tr>
<tr>
<td>HY</td>
<td>HY-1</td>
<td>58.90</td>
<td>59.13</td>
<td>0.0033</td>
<td>0.0034</td>
<td>0.37</td>
<td>0.306</td>
</tr>
<tr>
<td></td>
<td>HY-2</td>
<td>58.90</td>
<td></td>
<td>0.0037</td>
<td></td>
<td>0.31</td>
<td></td>
</tr>
<tr>
<td></td>
<td>HY-3</td>
<td>59.60</td>
<td></td>
<td>0.0033</td>
<td></td>
<td>0.24</td>
<td></td>
</tr>
</tbody>
</table>
The energy absorption capacity of SFR-HSC (MI, MA and HY) cylinders was calculated by considering the area under the stress-strain curve until a strain of 0.01. However, the energy absorption capacity of the HSC cylinders was measured until the strain corresponding to the maximum compressive stress due to brittle failure of the HSC. The capacity of the HSC to absorb energy significantly increased with the inclusion of steel fibres. A higher improvement in the energy absorption capacity of SFR-HSC was found with MI cylinders as shown in Table 7.1. It was found that the average energy absorption capacity of MI cylinders increased by 2.2 times compared to HSC cylinders. However, the average energy absorption capacity of MA and HY cylinders increased by 1.2 and 1.95 times, respectively, compared to HSC cylinders.

Figure 7.3 shows the energy absorption capacity of HSC, MI, MA and HY before and after maximum compressive stress. It can be seen from Figure 7.3 that the HSC does not absorb any energy after maximum compressive stress due to brittle failure of the HSC cylinders. However, the HSC reinforced with steel fibres have absorbed quite a significant amount of energy after maximum compressive stress. It was found from Figure 7.3 that the amount of energy absorbed by MI and HY cylinders before and after maximum compressive stress was higher than the amount of energy absorbed by MA and HSC cylinders.
7.2.2 Compressive Strength of the Cylinder

From the ready-mix HSC, a total of twelve cylinders of 100 mm x 200 mm were cast and tested to determine the compressive strength of HSC, MI, MA and HY cylinders. The typical failure modes of the cylinders tested under compression are shown in Figure 7.4. It was observed that the HSC cylinders failed in a brittle manner. However, with the inclusion of steel fibres into HSC, the failure modes became a combination of shear failure and bulging of the cylinders in the lateral direction. Also, the failure mode of MI cylinders was due to the propagation of cracks parallel to the loading direction along with the bulging of the cylinders in the lateral direction. The failure modes of MA and HY cylinders were due to shear failure and bulging of the cylinders in the lateral direction as shown in Figure 7.4.
Table 7.2 presents the results of the compressive strength of HSC, MI, MA and HY cylinders. The compressive strength of HSC slightly improved with the inclusion of micro steel fibre. Whereas, the compressive strength of HSC reduced with the adding of macro steel fibres and hybrid steel fibres into HSC. It was observed that the average compressive strength increased by 2% of MI cylinders compared to HSC cylinder. However, the average compressive strength reduced by 13% and 2% of MA and HY cylinders, respectively, compared to HSC cylinder.

Figure 7.4. Typical failure modes for the specimens tested under a compression.
Table 7.2. Results of the compressive strength of HSC, MI, MA and HY cylinders.

<table>
<thead>
<tr>
<th>Group</th>
<th>Cylinder No.</th>
<th>Compressive strength (MPa)</th>
<th>Average of compressive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSC</td>
<td>HSC-1</td>
<td>63.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>HSC-2</td>
<td>66.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>HSC-3</td>
<td>65.9</td>
<td></td>
</tr>
<tr>
<td>MI</td>
<td>MI-1</td>
<td>67.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>MI-2</td>
<td>64.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>MI-3</td>
<td>68.0</td>
<td></td>
</tr>
<tr>
<td>MA</td>
<td>MA-1</td>
<td>57.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>MA-2</td>
<td>58.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>MA-3</td>
<td>54.3</td>
<td></td>
</tr>
<tr>
<td>HY</td>
<td>HY-1</td>
<td>63.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>HY-2</td>
<td>63.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>HY-3</td>
<td>64.1</td>
<td></td>
</tr>
</tbody>
</table>

7.2.3 Splitting Tensile Strength of the Cylinders

From ready-mix HSC, a total of twelve cylinders of 100 mm x 200 mm were cast and tested to determine the splitting tensile strength of HSC, MI, MA and HY cylinders. The typical failure modes of the cylinders tested under splitting tensile are shown in Figure 7.5. It was found that the HSC cylinders were split into two parts. However, the cylinders reinforced with steel fibres did not split under a tension load. The failure modes of steel fibre reinforced HSC cylinders were due to the propagation of cracks parallel to the loading direction. It was found that the width of cracks of MI and MA cylinders were about 15.5 mm and 5.8 mm, respectively. The difference in width of cracks of the MI and MA cylinders could be because of the short length of the micro steel fibres compared to the long length of the macro steel fibres. For HY cylinders the
width of crack was about 1.2 mm. This could be due to the benefit effect of micro and macro steel fibres.

Figure 7.5. Typical failure modes for the specimens tested under a splitting tensile.

Table 7.3 presents the results of the splitting tensile strength of HSC, MI, MA and HY cylinders. The splitting tensile strength of the HSC significantly improved with the inclusion of steel fibres. A higher improvement in the splitting tensile strength was found with the inclusion of hybrid steel fibres into HSC. It was observed that the average splitting tensile strength increased by 95%, 91%, and 105% of MI, MA and HY cylinders, respectively, compared to HSC cylinders.
Table 7.3. Results of the splitting tensile strength of HSC, MI, MA and HY cylinders.

<table>
<thead>
<tr>
<th>Group</th>
<th>Cylinder no.</th>
<th>Splitting tensile strength (MPa)</th>
<th>Average of splitting tensile strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSC</td>
<td>HSC-1</td>
<td>3.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>HSC-2</td>
<td>3.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>HSC-3</td>
<td>3.4</td>
<td>3.56</td>
</tr>
<tr>
<td>MI</td>
<td>MI-1</td>
<td>6.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>MI-2</td>
<td>7.0</td>
<td>6.96</td>
</tr>
<tr>
<td></td>
<td>MI-3</td>
<td>7.4</td>
<td></td>
</tr>
<tr>
<td>MA</td>
<td>MA-1</td>
<td>6.8</td>
<td>6.80</td>
</tr>
<tr>
<td></td>
<td>MA-2</td>
<td>6.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>MA-3</td>
<td>6.9</td>
<td></td>
</tr>
<tr>
<td>HY</td>
<td>HY-1</td>
<td>7.5</td>
<td>7.30</td>
</tr>
<tr>
<td></td>
<td>HY-2</td>
<td>7.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>HY-3</td>
<td>7.0</td>
<td></td>
</tr>
</tbody>
</table>

7.3 Results of the Main Experimental Program

In this section, the main experimental program results of four steel reinforced HSC specimens and twelve steel reinforced SFR-HSC specimens are presented. The response regarding axial load, corresponding axial and lateral deformations of the specimens and strain development in the longitudinal and helical reinforcement were recorded during testing of the specimens and reported herein. The behaviour of specimens tested under concentric axial load, 25 mm and 50 mm eccentric axial load and four-point bending are discussed and presented.

The ductility is one of the parameters used to investigate the performance of the specimens under different load conditions (Hadi 2009, Hadi and Widiarsa 2012). In this study, the ductility of the specimens ($\mu_{80}$) tested under concentric, 25 mm and 50 mm eccentric axial load was calculated as a ratio of the axial deformation at 80% of the
maximum axial load ($\Delta_{80}$) in the descending part of the axial load-axial deformation curves and the axial deformation at yield load ($\Delta_1$), as shown in Figure 7.6. To avoid any variation in the results of the ductility of the specimens, the yield load was assumed to be the load corresponding to an approximation of the limit of the elastic behaviour (Pessiki and Pieroni 1997, Hadi and Widiarsa 2012). In order to estimate the axial deformation at the limit of elastic behaviour, a best-fit line to the linear ascending branch of the axial load-axial deformation curve for each specimen was determined by linear regression analysis considering up to 80% of the maximum axial load. The line was then extrapolated to intersect with a horizontal line through the maximum axial load. The axial deformation corresponding to this intersection was defined as axial deformation at yield load (Pessiki and Pieroni 1997, Hadi and Widiarsa 2012).

Figure 7.6. Ductility measurement of the specimens tested under concentric, 25 mm and 50 mm eccentric axial load.
7.3.1 Failure Modes of the Specimens

All specimens were tested to failure. The failure of the specimens depends on the loading condition (concentric axial load, eccentric axial load and four-point bending) and the reinforcement materials. The failure modes of the specimens tested under concentric axial load are presented in Figure 7.7. For the specimens tested under concentric axial load, Specimen RC-0 failed in compression. The concrete cover spalled off from the whole section of Specimen RC-0 when the axial load reached their first peak. The failure mode of Specimen RC-0 was due to buckling of the longitudinal reinforcement and rupture of the helices. The inclined shear sliding surface separated the concrete core into two wedges. The failure mode of the specimens reinforced with steel fibres was different from the failure mode of the specimens without steel fibres. The inclusions of steel fibres into the specimens changed the failure mode of the specimens from brittle to ductile. Delay in the spalling off of the concrete cover was observed at first peak axial load for all the specimens reinforced with steel fibres. Specimen MI-0 failed by crushing of the concrete at the mid-height of the specimen due to the presence of small holes, which was followed by outward buckling of the longitudinal reinforcement and rupture of the helical reinforcement. Specimen MA-0 failed in compression. It was observed that the concrete cover spalled off, followed by buckling of the longitudinal reinforcement and fracture of helix steel reinforcement. Specimen HY-0 failed in compression, and the specimen expanded in the lateral direction. No cover spalling was observed at the first peak axial load of Specimen HY-0. The test stopped with the rupture of the helical steel bars.
Figure 7.8 presents the failure mode of the specimens tested under 25 mm eccentric axial load. Figure 7.9 shows the failure mode of the specimens tested under 50 mm eccentric axial load. The failure modes of Specimens RC-25 and RC-50 were marked by the loss of the concrete cover and buckling of the longitudinal bars on the compression side. The cracking of concrete in the horizontal direction was observed on the tension side. For Specimens MI-25 and MI-50, formation and development of horizontal cracks in the tension face and vertical cracks in the compression face followed by buckling of longitudinal reinforcement on the compression side were observed. For Specimen MA-25, the concrete cover was crushed on the compression side and cracked horizontally on the tension side. For Specimen MA-50, failure was noticed close to the top end of the specimen. This failure mode was not expected. The reason for that could be the loading roller on the top end of Specimen MA-50 was not aligned properly. No cover was spalled off on the compression side. Horizontal cracks were observed on the tension side.
side of Specimen MA-50. The failure modes of Specimens HY-25 and HY-50 were due to vertical cracks on the compression side and horizontal cracks on the tension side followed by hearing a loud sound.

Figure 7.8. Failure mode of the specimens tested under 25 mm eccentric axial load.

Figure 7.9. Failure mode of the specimens tested under 50 mm eccentric axial load.
For the specimens tested as beams under four-point bending, all specimens failed in bending. Figure 7.10 shows the failure mode of the specimens tested under four-point bending. For Specimen RC-B, cover spalling was observed in the compression zone while no cover spalling was observed in the compression zone for the specimens reinforced with steel fibres (MI-B, MA-B and HY-B). Besides, all the tests ended by the rupture of the longitudinal bars on the tension side.

Figure 7.10. Failure modes of the beam specimens tested under four-point bending.
7.3.2 Behaviour of Specimens Tested under Concentric Axial Load

A total of four specimens (RC-0, MI-0, MA-0 and HY-0) were tested under concentric axial load. Table 7.4 presents the results of the specimens tested under concentric axial load. The axial load-axial deformation curves of the specimens tested under concentric axial load are presented in Figure 7.11. In this figure, three points including first peak axial load ($P_{1p}$), axial load after cover spalling ($P_{cover}$) and axial load carried by concrete core (second peak load) ($P_{core}$) were considered for the purpose of comparison between the specimens.

The first peak axial load ($P_{1p}$) of the specimens was compared with the nominal axial load ($N_{uo}$) computed according to the AS 3600-09 (Australiain Standard 2009) as follows:

$$N_{uo} = \alpha_1 f'_c (A_c - A_{st}) + f_y A_{st} \quad (7.1)$$

where $f'_c$ is the unconfined concrete compressive strength, the factor $\alpha_1$ is equal to 1.0-0.003 $f'_c$ within the range of $0.72 \leq \alpha_1 \leq 0.85$, as recommended in AS 3600-09 (Australiain Standard 2009), $A_c$ is the total concrete cross-sectional area, $A_{st}$ is the cross-sectional area of the longitudinal bars, $f_y$ is the tensile yield stress of the longitudinal bar. For the specimens reinforced with steel fibres, the $f'_c$ is replaced with the compressive strength of unconfined SFR-HSC ($f'_c$). It was found that the ratio of $P_{1p}/N_{uo}$ ranged between 0.77 and 1.21. The higher ratios of $P_{1p}/N_{uo}$ correspond to Specimen HY-0. It was observed that Specimen MI-0 carried lower axial load than Specimen RC-0. This might be due to lack of adequate vibration which resulted in the formation of small holes in the specimen at the mid-height. The ratio of $P_{1p}/N_{uo}$ was 0.77, 0.96 and 1.21 for Specimens MI-0, MA-0 and HY-0, respectively. The axial
deformation of the specimens at the first peak axial load increased by 7%, 2% and 31% for Specimens MI-0, MA-0 and HY-0, respectively, compared to Specimen RC-0.

The axial load dropped to around 85% - 100% of first peak axial load due to cover spalling or crashing. The axial load of Specimens RC-0, MI-0 and HY-0 dropped by 85%, 98% and 91%, respectively. However, the axial load of Specimen MA-0 did not reduce. The axial load carried by the concrete at that point can be defined as $P_{cover}$. The value of $P_{cover}$ was calculated based on the reduction in the first peak axial load of each specimen. The $P_{cover}$ was compared with the unconfined compressive strength of the total concrete cross-sectional area ($A_c$), $N_{uoc} = \alpha_1 f'_c (A_c - A_{st})$. It was found that the ratio of $P_{cover}/N_{uoc}$ ranged from 0.91 to 1.41. Specimen HY-0 showed a higher ratio of $P_{cover}/N_{uoc}$ of 1.41. However, Specimen MI-0 showed a lower ratio of $P_{cover}/N_{uoc}$ of 0.91. The ratio of $P_{cover}/N_{uoc}$ was 1.19 and 1.0 for Specimens MA-0 and RC-0, respectively. It was observed that the ratio of $P_{cover}/N_{uoc}$ was larger than 1 for specimens reinforced with steel fibres, except Specimen MI-0, and equal to 1 for Specimen RC-0. This indicates that the inclusion of the steel fibres into HSC leads to delay the spalling of concrete cover. In this study, the drop in the first peak axial load was considered as a sign for delaying of the concrete cover. The axial deformation at the $P_{cover}$ of Specimens MI-0 and HY-0 increased by 1% and 22%, respectively. However, the axial deformation at the $P_{cover}$ for Specimen MA-0 decreased by 10% compared to Specimen RC-0.

Afterwards, the concrete core gained strength due to the effectiveness of confinement pressure provided by the helical reinforcement and steel fibres. Hence the axial load reached a second peak load ($P_{core}$), when the concrete core reached its maximum stress. The value of $P_{core}$ was calculated for each specimen by subtracting the load carried by the longitudinal bars from the second peak axial load. The axial load carried by the
longitudinal bars was calculated based on their cross-sectional area \( (A_{st}) \) and the stress-strain relationship of steel obtained from the tension test of the bars. The \( P_{\text{core}} \) was compared with the unconfined compressive strength of the total concrete core, \( N_{\text{uocc}} = \alpha_1 f'_c A_{cc} \), where \( A_{cc} \) is the cross-sectional area of the concrete core defined by the centreline of the helical bars. It was indicated that the ratio of \( P_{\text{core}}/N_{\text{uocc}} \) ranged from 1.34 to 1.94. Specimen HY-0 showed a higher ratio of \( P_{\text{core}}/N_{\text{uocc}} \) of 1.94. However, Specimen MI-0 showed a lower ratio of \( P_{\text{core}}/N_{\text{uocc}} \) of 1.34. The ratio of \( P_{\text{core}}/N_{\text{uocc}} \) was 1.58 and 1.51 for Specimens MA-0 and RC-0, respectively. Higher ratios were observed for well-confined specimens. Values higher than 1.51 indicate an improvement of strength because of the indirect confinement provided by steel fibres. For Specimens MA-0 and HY-0, the confinement effect increased by 8% and 29% compared to Specimen RC-0. However, for Specimen MI-0 the confinement decreased by 11% compared to Specimen RC-0. This is due to the small holes in the mid-height of Specimen MI-0. The axial deformation at the \( P_{\text{core}} \) increased by 15% and 16% for Specimens MA-0 and HY-0, respectively, compared to Specimen RC-0. However, the axial deformation of Specimen MI-0 decreased by 22% compared to Specimen RC-0.

The ductility of the tested specimens under concentric axial load increased with the inclusion of steel fibres. Specimen HY-0 exhibited the highest enhancement in ductility. The ductility of Specimens MI-0, MA-0 and HY-0 were increased by 19%, 16%, and 27%, respectively, compared to Specimen RC-0.

The axial strain of the longitudinal bars decreased with the inclusion of steel fibres. Appendix B presents the reading of strain gauges of the reinforcement bars of the specimens. It was found that the axial yield strain of the longitudinal bars at the first
peak axial load for Specimens MA-0 and HY-0 increased by 30% and 75%, respectively, compared to the axial yield strain of the tested longitudinal bars. However, the axial strain of the longitudinal bars reached their yield strain at the first peak axial load for Specimens RC-0 and MI-0. Also, the hoop strain of the helical bars increased with the inclusion of the steel fibres into the specimens. It was observed that the hoop strain of the helical bars at the first peak axial load for Specimen RC-0 increased by 11% compared to hoop strain of the tested helical bars. However, the hoop strain of the helical bars at the first peak axial load for Specimens MI-0, MA-0 and HY-0 increased by 122%, 11% and 67%, respectively, compared to the hoop strain of the tested helical bars.

Table 7.4. Results of the specimens tested under concentric axial loads.

<table>
<thead>
<tr>
<th>Specimens</th>
<th>RC-0</th>
<th>MI-0</th>
<th>MA-0</th>
<th>HY-0</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_{1p}$ (kN)</td>
<td>1827</td>
<td>1441</td>
<td>1626</td>
<td>2196</td>
</tr>
<tr>
<td>$P_{1p}/N_{uo}$</td>
<td>0.99</td>
<td>0.77</td>
<td>0.96</td>
<td>1.21</td>
</tr>
<tr>
<td>$P_{cover}$ (kN)</td>
<td>1553</td>
<td>1412</td>
<td>1626</td>
<td>2018</td>
</tr>
<tr>
<td>$P_{cover}/N_{uoc}$</td>
<td>1.0</td>
<td>0.91</td>
<td>1.19</td>
<td>1.41</td>
</tr>
<tr>
<td>$P_{core}$ (kN)</td>
<td>1601</td>
<td>1412</td>
<td>1721</td>
<td>2066</td>
</tr>
<tr>
<td>$P_{core}/N_{uocc}$</td>
<td>1.51</td>
<td>1.34</td>
<td>1.58</td>
<td>1.94</td>
</tr>
</tbody>
</table>

Axial deformation (mm)

<table>
<thead>
<tr>
<th></th>
<th>Corresponding to $P_{1p}$</th>
<th>Corresponding to $P_{cover}$</th>
<th>Corresponding to $P_{core}$</th>
<th>Ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC-0</td>
<td>2.57</td>
<td>2.98</td>
<td>4.64</td>
<td>3.25</td>
</tr>
<tr>
<td>MI-0</td>
<td>2.74</td>
<td>3.00</td>
<td>3.62</td>
<td>3.88</td>
</tr>
<tr>
<td>MA-0</td>
<td>2.62</td>
<td>2.68</td>
<td>5.33</td>
<td>3.77</td>
</tr>
<tr>
<td>HY-0</td>
<td>3.36</td>
<td>3.64</td>
<td>5.39</td>
<td>4.13</td>
</tr>
</tbody>
</table>

where $P_{1p}$ is the first peak axial load, $N_{uo}$ is nominal axial load, $P_{cover}$ is axial load carried by the concrete cover, $N_{uoc}$ is the compressive strength of the total concrete cross-sectional area, $P_{core}$ is second peak load, $N_{uocc}$ is the compressive strength of the total concrete core.
7.3.3 Behaviour of Specimens Tested under Eccentric Axial Load

A total of eight specimens were tested under eccentric axial load. Specimens RC-25, MI-25, MA-25 and HY-25 were tested under 25 mm eccentric axial load and Specimens RC-50, MI-50, MA-50 and HY-50 were tested under 50 mm eccentric axial load. Table 7.5 presents the results of the specimens tested under 25 mm eccentric axial loads. Axial load-axial deformation and axial load-lateral deformation curves of the specimens tested under 25 mm eccentric axial load are presented in Figure 7.12. These curves consist of two parts, i.e., an ascending part and a descending part. The ascending part of the curves is almost the same until the maximum axial load. The influence of the steel fibres was observed in the descending part of the curves. Higher improvement in the maximum axial load and corresponding axial and lateral deformations was observed for Specimen MI-25. The maximum axial load of Specimen MI-25 was increased by 28% compared
to Specimen RC-25. The axial and lateral deformations at the maximum axial load of Specimen MI-25 were increased by 22% and 82%, respectively, compared to Specimen RC-25. For Specimen MA-25, the maximum axial load was increased by 9% compared to Specimen RC-25. The axial and lateral deformations at the maximum axial load of Specimen MA-25 were almost the same as those of Specimen RC-25. The maximum axial load of Specimen HY-25 was increased by 11% compared to Specimen RC-25. Also, the axial and lateral deformations at the maximum axial load of Specimen HY-25 were increased by 12% and 10%, respectively, compared to Specimen RC-25. The inclusion of steel fibres led to improvement in ductility of the specimens. Specimen HY-25 had the highest improvement in ductility. The ductility was increased by approximately 20%, 35% and 53% for Specimens MI-25, MA-25, and HY-25, respectively, compared to Specimen RC-25.

Table 7.5. Results of the specimens tested under 25 mm eccentric axial load.

<table>
<thead>
<tr>
<th>Specimens</th>
<th>RC-25</th>
<th>MI-25</th>
<th>MA-25</th>
<th>HY-25</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial load (kN)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yield</td>
<td>1140</td>
<td>1454</td>
<td>1115</td>
<td>1168</td>
</tr>
<tr>
<td>Maximum</td>
<td>1140</td>
<td>1464</td>
<td>1248</td>
<td>1270</td>
</tr>
<tr>
<td>Deformation (mm)</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>At yield load</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Axial</td>
<td>2.37</td>
<td>2.69</td>
<td>2.40</td>
<td>2.16</td>
</tr>
<tr>
<td>Lateral</td>
<td>1.48</td>
<td>2.27</td>
<td>0.95</td>
<td>1.01</td>
</tr>
<tr>
<td>At maximum load</td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Axial</td>
<td>2.37</td>
<td>2.89</td>
<td>2.53</td>
<td>2.65</td>
</tr>
<tr>
<td>Lateral</td>
<td>1.48</td>
<td>2.70</td>
<td>1.13</td>
<td>1.62</td>
</tr>
<tr>
<td>Bending moment (kN.m)</td>
<td>30.19</td>
<td>39.92</td>
<td>32.39</td>
<td>33.03</td>
</tr>
<tr>
<td>Ductility</td>
<td>1.65</td>
<td>1.98</td>
<td>2.23</td>
<td>2.52</td>
</tr>
</tbody>
</table>
Figure 7.12. Axial load-axial deformation and axial load-lateral deformation curves of specimens tested under 25 mm eccentric axial load.

Table 7.6 presents results of the specimens tested under 50 mm eccentric axial loads. Axial load-axial deformation and axial load-lateral deformation curves of the specimens tested under 50 mm eccentric axial load are presented in Figure 7.13. It was observed that the highest improvements in the maximum axial load and corresponding deformations were observed for Specimen MI-50. The maximum axial load of Specimen MI-50 was increased by 46% compared to Specimen RC-50. The axial and lateral deformations at the maximum axial load of Specimen MI-50 were increased by 30% and 88%, respectively, compared to Specimen RC-50. For Specimen MA-50, the maximum axial load was increased by 22% compared to Specimen RC-50. The axial deformation at the maximum axial load of Specimen MA-50 and Specimen RC-50 was similar, whereas the lateral deformation was increased by 13% for Specimen MA-50.
compared to Specimen RC-50. The maximum axial load of Specimen HY-50 was increased by 23% compared to Specimen RC-50. The axial and lateral deformations at the maximum axial load of Specimen HY-50 were increased by 2% and 14%, respectively, compared to Specimen RC-50. The higher improvement in ductility was observed for Specimen HY-50. The enhancement in ductility was 33%, 63% and 65% for Specimens MI-50, MA-50 and HY-50, respectively, compared to Specimen RC-50.

Table 7.6. Results of the specimens tested under 50 mm eccentric axial load.

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Axial load (kN)</th>
<th>Deformation (mm)</th>
<th>Bending moment (kN.m)</th>
<th>Ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Yield</td>
<td>Maximum</td>
<td>At yield load Axial</td>
<td>2.11</td>
</tr>
<tr>
<td>RC-50</td>
<td>661</td>
<td>697</td>
<td>1.83</td>
<td>2.25</td>
</tr>
<tr>
<td>MI-50</td>
<td>1000</td>
<td>1021</td>
<td>2.78</td>
<td>3.25</td>
</tr>
<tr>
<td>MA-50</td>
<td>718</td>
<td>848</td>
<td>2.12</td>
<td>1.97</td>
</tr>
<tr>
<td>HY-50</td>
<td>803</td>
<td>857</td>
<td>2.13</td>
<td>1.97</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>At maximum load Axial</td>
<td>2.39</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.39</td>
<td>3.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.44</td>
<td>2.39</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Lateral</td>
<td>2.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.55</td>
<td>4.22</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.56</td>
<td>2.55</td>
</tr>
</tbody>
</table>
From results of the specimens tested under 25 mm and 50 mm eccentric axial load, it was indicated that Specimens MI-25 and MI-50 showed higher enhancement in the maximum axial load and corresponding deformations (axial and lateral deformations). This could be due to the lower aspect ratio (length/diameter) of micro steel fibres and the higher percentage of the fibres (3%), which provides a larger number of fibres in the concrete cross-section. Hence, the micro steel fibres produced more effective reinforcing mechanisms at the microcracking level (early stage). This resulted in an increase in the maximum axial load. However, the descending part of Specimens MI-25 and MI-50 was steeper compared to Specimens MA-25, MA-50, HY-25 and HY-50. This could also be because of the aspect ratio of micro steel fibres, where the fibres were pulled out from the matrix and were ineffective in resisting the axial load. This
might be one of the reasons for the lower improvement in the ductility for Specimen MI compared to Specimens MA and HY.

Specimens MA-25 and MA-50 showed higher improvement in the ductility compared to the improvement in the maximum axial load. This might be because of the higher aspect ratio (length/diameter) of macro steel fibres and volume content of 2% of the fibres, which provided less numbers of the fibres in the cross-section of the specimens. Thus, the macro steel fibres were not effective in delaying the appearance of microcracks at an early stage of applying the load. However, with the widening of the cracks, macro steel fibres became effective and delayed further widening of the cracks by carrying stresses and transferring the stresses to other parts of the matrix or other fibres.

For Specimens HY-25 and HY-50, both the maximum axial load and the ductility were improved. The reason of these improvements could be the combined beneficial effect of micro steel fibres and macro steel fibres. The inclusion of micro steel fibres arrested microcracks and the addition of macro steel fibres enhanced the post-cracking behaviour by arresting wider cracks. This could also be the reason for their higher improvement in maximum axial load and ductility.

7.3.4 Behaviour of Specimens Tested under Four-Point Bending

Four specimens were tested under four-point bending to determine the bending capacity. Table 7.7 presents the results of the specimens tested under four-point bending. The behaviour of Specimens RC-B, MI-B, MA-B and HY-B under four-point bending are shown in Figure 7.14. It can be seen from Figure 7.14 that the bending load
was significantly increased with the addition of steel fibres compared to Specimen RC-B.

The bending yield load and corresponding midspan deflection, bending maximum load and corresponding midspan deflection, and ductility of the specimens reinforced with steel fibres under four-point bending were compared to those of Specimen RC-B. The strain gauges on the longitudinal bars of the specimens tested under four-point loading were damaged. Thus, to avoid any variation in the results of the ductility, the yield load was assumed to be the load corresponding to an approximation of the limit of the elastic behaviour (Pessiki and Pieroni 1997, Hadi and Widiarsa 2012).

The highest improvement in the bending yield load and corresponding midspan deflection was found for Specimen HY-B. It can be seen that the bending yield load increased by about 17%, 8% and 35% for Specimens MI-B, MA-B and HY-B, respectively, compared to Specimen RC-B. The midspan deflection at bending yield load was increased by 6% and 34% for Specimens MI-B and HY-B, respectively, compared to Specimen RC-B. However, the midspan deflection at bending yield load of Specimen MA-B was decreased by 20% compared to Specimen RC-B.

The highest enhancement in the bending maximum load and corresponding midspan deflection was observed for Specimen HY-B. It was indicated that the bending maximum load increased by 17%, 11%, 35% for Specimens MI-B, MA-B and HY-B, respectively, compared to Specimen RC-B. The midspan deflection corresponding to bending maximum load increased by 17%, 8%, and 36% for Specimens MI-B, MA-B and HY-B, respectively, compared to Specimen RC-B.
The ductility of the specimens tested under four-point bending was determined as a ratio of the midspan deflection at failure (rupture of the longitudinal bar) in the descending part of the bending load-midspan deflection curves and the midspan deflection at the bending yield load. It was found that the ductility of the specimens under four-point bending was considerably improved by the inclusion of steel fibres. Specimen MA-B showed the highest improvement in the ductility. The ductility increased by about 12%, 75% and 11% for Specimens MI-B, MA-B, and HY-B, respectively compared to Specimen RC-B. The reason for the highest ductility of Specimen MA-B could be attributed to the high percentage of macro steel fibres (2%) in Specimen MA-B.

Table 7.7. Results of the specimens tested under four-point bending.

<table>
<thead>
<tr>
<th>Specimens</th>
<th>RC-50</th>
<th>MI-50</th>
<th>MA-50</th>
<th>HY-50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending load (kN)</td>
<td>Yield</td>
<td>236</td>
<td>277</td>
<td>255</td>
</tr>
<tr>
<td></td>
<td>Maximum</td>
<td>294</td>
<td>344</td>
<td>327</td>
</tr>
<tr>
<td>Midspan deflection (mm)</td>
<td>Yield</td>
<td>4.84</td>
<td>5.13</td>
<td>3.87</td>
</tr>
<tr>
<td></td>
<td>At maximum load</td>
<td>23.70</td>
<td>28.27</td>
<td>33.20</td>
</tr>
<tr>
<td>Bending moment (kN.m)</td>
<td>27.73</td>
<td>32.55</td>
<td>29.96</td>
<td>37.60</td>
</tr>
<tr>
<td>Ductility</td>
<td>4.90</td>
<td>5.50</td>
<td>8.57</td>
<td>5.46</td>
</tr>
</tbody>
</table>
7.3.5 Influence of Eccentricity on the Behaviour of the Specimens

7.3.5.1 Group RC Tested under Axial Compression

The axial load-axial deformation curves of Specimens RC-0, RC-25 and RC-50 are shown in Figure 7.15. It can be seen from Figure 7.15 that the first peak axial load of Specimen RC-0 was much higher than the maximum axial load of Specimens RC-25 and RC-50. The first peak axial load of Specimen RC-0 was higher by 1.6 and 2.6 times than the maximum axial load of Specimens RC-25 and RC-50, respectively. The ductility of the specimens decreased with the increase of the eccentricities. The ductility of Specimens RC-25 and RC-50 decreased by 49% and 53% compared to Specimen RC-0, while the ductility of Specimen RC-50 decreased by 7% compared to Specimen RC-25.
7.3.5.2 Group MI Tested under Axial Compression

The axial load-axial deformation of Specimens MI-0, MI-25 and MI-50 are presented in Figure 7.16. It can be seen from Figure 7.16 that the first peak axial load of Specimen MI-0 was slightly lower than the maximum axial load of Specimen MI-25. This is because the small holes in the mid-height of Specimen MI-0 which led to reducing the axial load of Specimen MI-0. The first peak axial load of Specimen MI-0 decreased by 2% compared to the maximum axial load of Specimen MI-25. The maximum axial load decreased with the increase of the eccentric load. The maximum axial load of Specimen MI-50 decreased by 30% compared to Specimen MI-25. The ductility of the specimens decreased with the increase of the eccentricity. The ductility decreased by 49% and 48% for Specimens MI-25 and MI-50, respectively, compared to Specimen MI-0.
7.3.5.3 Group MA Tested under Axial Compression

The axial load-axial deformation of Specimens MA-0, MA-25 and MA-50 are presented in Figure 7.17. It can be seen from Figure 7.17 that the first peak axial load of Specimen MA-0 was much higher than the maximum axial load of Specimens MA-25 and MA-50. The maximum axial load of Specimens MA-25 and MA-50 decreased by 38% and 62% compared to the first peak axial load of Specimen MA-0. The ductility of the specimens decreased with the increase of the eccentricity. The ductility of Specimens MA-25 and MA-50 decreased by 41% and 33% compared to Specimen MA-0. However, the ductility of Specimen MA-50 increased by 14% compared to Specimen MA-25.
7.3.5.4 Group HY Tested under Axial Compression

The axial load-axial deformation of Specimens HY-0, HY-25 and HY-50 are presented in Figure 7.18. It can be seen from Figure 7.18 that the first peak axial load of Specimen HY-0 was much higher than the maximum axial load of Specimens HY-25 and HY-50. In addition, the maximum axial load of Specimens HY-25 and HY-50 decreased by 42% and 61% compared to the first peak axial load of Specimen HY-0. The ductility of the specimens decreased with the increase of the eccentricity. The ductility of both Specimens HY-25 and HY-50 decreased by 39% compared to Specimen HY-0. However, the ductility of Specimen HY-25 and HY-250 were the same.
7.3.6 Experimental Axial Load-Bending Moment Interaction Diagrams of the Specimens

In this study, experimental load-moment \((P-M)\) interaction diagrams were drawn to investigate the axial load and bending moment capacities of the specimens. The \(P-M\) interaction diagram was drawn based on four points, \(i.e.,\) concentric axial load, 25 mm and 50 mm eccentric axial load, and four-point bending. The experimental bending moment for the specimens tested under different eccentricities, including the secondary moment, was determined by multiplying the maximum axial load with the summation of eccentricity and lateral deformation at the maximum axial load according to Equation 7.2.

\[
M = P_{\text{max}}(e + \Delta)
\]
where, $M$ refers to the bending moment at the mid-height of the specimens, $P_{\text{max}}$ refers to the maximum axial load carried by the specimens, $e$ refers to the eccentricity (25 mm or 50 mm), $\Delta$ refers to the lateral deformation at maximum axial load. For the specimens tested under four-point bending, the bending moment ($M_B$) was determined according to Equation 7.3 as follows:

$$M_B = \frac{P_{\text{yield}}}{6}L$$

(7.3)

where $L$ is the span length between the two supports. In this study, $L$ was taken as 705 mm. The experimental bending moment for the specimens tested under 25 mm and 50 mm eccentric axial load was shown in Table 7.5 and Table 7.6, respectively. The experimental bending moment for specimens tested under four-point bending was shown in Table 7.7. Figure 7.19 shows the $P-M$ interaction diagrams for the tested specimens. It can be seen from Figure 7.19 that the capacity of the specimens in terms of the axial load and bending moment significantly improved with the inclusion of steel fibres. Specimens MI-25 and MI-50 showed higher axial load and bending moment compared with the other specimens tested under 25 mm and 50 mm eccentric axial load. However, Specimen MI-B sustained lower bending moment than Specimen HY-B and higher bending moment than Specimen MA-B. It can be seen from Figure 7.19 that Specimens MA-25 and MA-50 showed almost the same improvement in the axial load and bending moment capacity as Specimens HY-25 and HY-50. However, Specimens MA-B showed lower enhancement in the bending moment capacity than Specimens MI-B and HY-B. Specimens HY also sustained a higher axial load and bending moment under different eccentric axial loads and four-point bending compared to Specimens
MA and RC. Specimen HY-B showed a higher improvement in the bending moment than the other specimens tested under four-point bending.

Figure 7.19. Experimental $P$-$M$ interaction diagram of the tested specimens.

7.3.7 Experimental Bending Moment-Curvature Responses of the Specimens

The bending moment-curvature ($M$-$\phi$) curves were drawn for the specimens tested under eccentric axial load. The experimental bending moment at mid-height of the tested specimens under eccentric axial load was determined as shown in Equation 7.2. The experimental curvature ($\phi$) corresponding to the bending moment was determined using the strain gauges reading at the longitudinal bars as follows:

$$\phi = \frac{(\varepsilon_{sc} - \varepsilon_{st})}{d_c}$$

(7.4)
where $\varepsilon_{sc}$ is the strain gauge readings of the longitudinal bar on the compression side, $\varepsilon_{st}$ is the strain gauges reading on the longitudinal bar on the tension side, $d_c$ is the distance between two strain gauges as shown in Figure 6.7. In this study, the compression strain of the longitudinal bar was taken as a positive and the tension strain of the longitudinal bar was taken as a negative.

Figure 7.20 presents the experimental $M-\phi$ response of the specimens tested under 25 mm eccentric axial load. It can be seen from Figure 7.20 that. The higher improvement in the maximum bending moment was observed for Specimen MI-25. Specimen MA-25 showed a sudden reduction in the bending moment after reaching their maximum. This may be due to concrete cover spalling off on the compression side of the specimen. Also, higher improvement in the curvature corresponding to the maximum bending moment was observed for Specimen MI-25. The curvature corresponding to the maximum bending moment increased by 35% and 34% for Specimens MI-25 and HY-25, respectively, compared to Specimen MA-25.

Figure 7.21 shows the experimental $M-\phi$ response of the specimens tested under 50 mm eccentric axial load. Higher improvement in the bending moment was observed with Specimen MI-50 compared to Specimens MA-50 and HY-50. The curvature of Specimen MI-50 significantly improved with the increasing the applied axial load. However, the curvature of Specimen MA-50 was lower compared to Specimens MI-50 and HY-50. Also, the curvature corresponding to the maximum bending moment increased by 40% and 15% for Specimens MI-50 and HY-50, respectively, compared to Specimen MA-50.
Figure 7.20. Experimental bending $M$-$\phi$ curves of the specimens tested under 25 mm eccentric axial load.

Figure 7.21. Experimental bending $M$-$\phi$ curves of the specimens tested under 50 mm eccentric axial load.
7.4 Summary

In this chapter the results of the preliminary material program including compression test and splitting tensile test were reported. Also, the results of the main experimental program were presented. The experimental results including axial load, bending moment, axial deformation, lateral deformation and ductility of the specimens were presented. Also, the axial load-bending moment interaction diagrams and bending moment-curvature response of the specimens were explained. The results were discussed in order to demonstrate the influence of the different types of steel fibres on the behaviour of HSC specimens tested under different loading conditions. In the next chapter, the comparison between the experimental and analytical results is presented and discussed.
8: ANALYSIS OF EXPERIMENTAL RESULTS

8.1 General

This chapter presents the analytical results of the preliminary testing program and the main experimental program. The analytical results of the preliminary testing program consisted of constructing compressive stress-strain curves of High Strength Concrete (HSC) and Steel Fibre Reinforced High Strength Concrete (SFR-HSC) cylinders. The analytical results of the main experimental program consisted of two parts. The first part consisted of constructing axial load-bending moment interaction diagrams of Groups RC, MI, MA and HY by using two different analysis methods (equivalent rectangular stress block method and layer by layer integration method). The second part consisted of constructing bending moment-curvature curves using a layer by layer integration method for Groups MI, MA and HY tested under 25 mm and 50 mm eccentric axial loads. In addition, a parametric study was conducted to investigate the influence of different volume contents and aspect ratios of steel fibres on the axial load-bending moment interaction diagrams. The details of the analysis are explained in the following sections.

8.2 Stress-Strain Behaviour of the HSC and SFR-HSC Cylinders

In this study, the compressive stress-strain behaviour of the HSC and SFR-HSC (MI, MA and HY) cylinders was analytically modelled using available compressive stress-strain model. The stress-strain model proposed by Carriera and Chu (1985) (Equation 8.1) was used to construct the analytical stress-strain curves of the HSC, MI, MA and HY and is expressed as follows:
$$f_c = f'_c \left[ \frac{\beta (\varepsilon_c/\varepsilon'_c)}{\beta - 1 + (\varepsilon_c/\varepsilon'_c)^\beta} \right]$$  

(8.1a)

$$\beta = \frac{1}{1 - (f'_c / \varepsilon'_c / E_{it})}$$  

(8.1b)

where $f'_c$ is the maximum axial compressive stress of unconfined concrete, $\varepsilon'_c$ is the axial strain corresponding to $f'_c$, $\varepsilon_c$ is the axial strain at any compressive stress of the concrete, $\beta$ is the material parameter which depends on the shape of the stress-strain curve, and $E_{it}$ is the initial tangent modulus of concrete. For SFR-HSC (MI, MA and HY) cylinders, the parameters of $f'_c$, $\varepsilon'_c$ and $\beta$ above are replaced with $f_{ct}'$, $\varepsilon_{ct}'$ and $\beta_1$ respectively, in order to consider the contribution of steel fibres on the compressive stress of SFR-HSC, where $f_{ct}'$, $\varepsilon_{ct}'$ and $\beta_1$ refer to the maximum axial compressive stress, corresponding strain and material parameter, respectively, of SFR-HSC. The $\beta_1$ can be determined using the proposed model by Ou et al. (2011) as follows:

$$\beta_1 = 0.71 \,(R.I.)^2 - 2(R.I.) + 3.05$$  

(8.2)

where $R.I.$ is the reinforced index of steel fibres and is equal to $v_f \times l_f/d_f$, where $v_f$ is the volume content of steel fibres, $l_f/d_f$ is the aspect ratio of steel fibres $l_f$ is the length of the steel fibres and $d_f$ is the diameter of the steel fibres. Figure 8.1 presents the comparison between experimental and analytical stress-strain curves of the HSC and SFR-HSC cylinders. In general, it was found that the analytical stress-strain curves of the cylinders were in good agreement with the experimental results.
Figure 8.1. Experimental and Analytical results of stress-strain curves of (a) HSC, (b) MI, (c) MA and (d) HY cylinders.
8.3 Analytical Consideration for the Main Experimental Program

The analytical results of the main experimental program included two parts. The first part included constructing axial load-bending moment ($P$-$M$) interaction diagrams for Groups RC, MI, MA and HY. The second part included constructing bending moment-curvature ($M$-$\phi$) diagrams for the specimens tested under 25 mm and 50 mm eccentric axial loads. The section analysis of the specimen cross-section was carried out to calculate the analytical $P$-$M$ interaction diagrams for Groups RC, MI, MA and HY using Equivalent Rectangular Stress Block (ERSB) method and Layer by Layer (L-L) integration method. In addition, the analytical $M$-$\phi$ diagrams were calculated for the specimens tested under 25 mm and 50 mm eccentric axial loads using L-L integration method. The cross-section of the specimens was analysed by considering the following assumptions:

(i) Plane sections remain plane after deformation;
(ii) The tensile strength of the specimens reinforced with steel fibres was considered. However, the tensile strength of the specimens without steel fibres was ignored;
(iii) Perfect bond exists between steel reinforcement and surrounding concrete;
(iv) Elastic perfectly plastic behaviour was assumed for the stress-strain behaviour of longitudinal steel reinforcement.

In the following sections, the analytical procedures used in this study to analyse the cross-section area of the specimens are explained in detail.
8.4 Analytical P-M Interaction Diagrams

The analytical P-M interaction diagrams were drawn using two analytical methods i.e. ERSB method and L-L integration method. The ERSB is an analysis method recommended by ACI 318-14 (ACI 2014). In this method, the strength of the concrete cross-section was calculated by considering a uniform stress distribution. In the L-L integration method, the strength of concrete cross-section was determined by dividing the cross-section into a small number of layers. Also, the stress distribution in the L-L integration method is nonlinear and is based on the stress-strain responses of the concrete.

The longitudinal reinforcement in the specimens was located in four levels at a distance $d_x$ from the top extreme fibre compression. Based on similar triangles in strain distribution diagram, the strain in each steel bar ($\varepsilon_{sx}$) can be calculated as follows:

$$\varepsilon_{sx} = \left(1 - \frac{d_x}{d_n}\right) \varepsilon_{cu}$$

where $\varepsilon_{cu}$ is the ultimate compressive strain in the extreme compression fibre, $d_n$ is the depth of neutral axis of cross-section, $d_x$ is the distance between the centre of the longitudinal steel bar to the extreme compression fibre in the compression side. The force ($F_{sx}$) and bending moment ($M_{sx}$) in each longitudinal steel bar can be calculated as:

$$F_{sx} = \varepsilon_{sx} E_s A_{sx}$$

$$M_{sx} = \sum_{i=1}^{k} F_{sx} \left(\frac{D}{2} - d_x\right)$$
where $E_s$ is the modulus of elasticity of the longitudinal steel, $A_{sx}$ is the area of the longitudinal steel, $D$ is the diameter of the circular cross-section and $k$ is the number of the longitudinal steel bars.

The nominal axial load ($N_{uo}$) of the specimens tested under concentric axial load was calculated using Equation 7.1. Based on the principles equilibrium and strain compatibility between steel reinforcement and concrete in the concrete cross-section, the nominal axial load and bending moment capacities of Groups RC, MI, MA and HY tested under 25 mm and 50 mm eccentric axial load and four-point bending were calculated using the ERSB method and L-L integration method as discussed below.

8.4.1 The ERSB Method

The ERSB method as illustrated in ACI 318-14 (ACI 2014) was used to determine the strength of concrete in compression and tension sides. A uniform stress distribution was considered to calculate the strength of the concrete cross-section. The concrete in the cross-section of the specimens was considered as an unconfined concrete to simplify the calculation (ACI 2014).

In compression zone of ERSB as shown in Figure 8.3, the width ($\alpha f_c')$ and depth ($\gamma d_n$) of the ERSB were assumed to estimate the strength of the concrete. The effect of the steel fibres on the concrete in the compression zone was represented by replacing the unconfined concrete compressive strength ($f_c'$) with unconfined SFR-HSC compressive strength ($f_{ct}'$). Based on recommendation of ACI 544.4R-09 (ACI 2009), the ultimate compressive strain of SFR-HSC was assumed to be 0.004. The parameters of width ($\alpha$)
and depth ($\gamma$) of the ERSB in the compression zone can be calculated based on recommendation of AS 3600-09 (Australian Standard 2009) as follows:

$$\alpha = 1.0 - 0.003 f'_c \quad 0.72 \leq \alpha \leq 0.85 \quad (8.6a)$$

$$\gamma = 1.05 - 0.007 f'_c \quad 0.67 \leq \gamma \leq 0.85 \quad (8.6b)$$

In the tension zone of the ERSB, the effect of steel fibres was considered in SFR-HSC to carry tensile loads. Two different calculation procedures were adopted to calculate the tensile load carried by steel fibre in the tension zone of the ERSB. In the first calculation procedure, the contribution of the steel fibres to carry tensile stress was calculated using Equation 8.7 recommended in ACI 544.4R-09 (ACI 2009).

$$\sigma_{t1} = 0.00772 \frac{\ell_f}{d_f} v_f F_b \quad (8.7)$$

where $\sigma_{t1}$ is the tensile stress of steel fibre reinforced concrete, $F_b$ is the bond efficiency of the fibres which is taken as 1.2 in this study. In the second calculation procedure, the contribution of the steel fibres to carry tensile stress was determined using Equation 8.8 proposed by Bentur and Mindess (2006).

$$\sigma_{t2} = \eta \tau_f \frac{\ell_f}{d_f} v_f \quad (8.8)$$
where $\sigma_{t2}$ is the tensile strength of steel fibre reinforced concrete, $\eta_\theta$ is the orientation effectiveness factor of the fibres. This factor can be taken as 0.5 by considering the orientation of the fibres at $\theta \geq \pi/6$ (Maage 1977), $\tau_f$ is the bond shear strength of the fibres and is equal to $0.6(f'_c)^{2/3}$ (Marti et al. 1999).

Although the ERSB method usually deals with rectangular sections, it is applicable to circular sections. The effective unconfined concrete cross-section area ($A_{c,t}$) of equivalent rectangular cross section (as shown in Figure 8.2) can be calculated as follows:

$$A_{c,t} = D^2 \left( \frac{\theta_{(c,t)} - \sin \theta_{(c,t)} \cdot \cos \theta_{(c,t)}}{4} \right)$$  \hspace{1cm} (8.9a)

where $\theta_{(c,t)}$ is expressed in radius, Equations (8.9b) and (8.9c) can be used to calculate the $\theta_{(c,t)}$ for the compression and tension zones, respectively, $D$ in the diameter of the cross-section:

$$\theta_c = \cos^{-1} \left[ \frac{D/2 - \gamma d_n}{D/2} \right]$$  \hspace{1cm} (8.9b)

$$\theta_t = \cos^{-1} \left[ \frac{\gamma d_n - D/2}{D/2} \right]$$  \hspace{1cm} (8.9c)
Figure 8.2. Centroid of compression and tension zone of the circular cross-section.

Figure 8.3 shows the distribution of stress and strain for the circular cross-section. The concrete compressive force ($C_c$) acting at the centroid of the compression zone at a distance ($X_c'$) from the centre of cross-section (see Figure 8.2) was calculated as follows:

\[
C_c = \alpha f_c' A_c \quad \text{(8.10a)}
\]

\[
X_c' = \frac{D^3}{A_c} \left[ \frac{\sin^3(\theta_c)}{12} \right] \quad \text{(8.10b)}
\]

The concrete tensile force ($T_t$) acting at the centroid of the tension zone at a distance ($X_t'$) from the centre of the cross-section (see Figure 8.2) can be determined as follows:

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\[ T_t = \sigma_{t(1,2)} A_t \quad \text{(8.11a)} \]

\[ X'_t = \frac{D^3}{A_t} \left[ \sin^3(\theta_t) \right] \quad \text{(8.11b)} \]

where \( \sigma_{t1} \) and \( \sigma_{t2} \) are calculated using Equation (8.7) and Equation (8.8), respectively.

The bending moment can be calculated, by taking forces around the centroid of the cross-section of the specimen for the concrete and steel bars in compression and tension zones, as follows:

\[ M_n = C_c \* X'_c + T_t \* ((D - d_n/2) + X'_t) + \sum_{i=1}^{k} F_{sx} \left( \frac{D}{2} - d_x \right) \quad \text{(8.12)} \]
Figure 8.3. Distribution of stress and strain in the circular cross-section.
8.4.2. Analytical P-M Interaction Diagrams Using ERSB

Figure 8.4 shows the P-M interaction diagram of the experimental and analytical results of Group RC. It can be seen from Figure 8.4 that the analytical results of P-M interaction diagram of Group RC underestimated the corresponding experimental results. Analytical axial load of Specimens RC-0, RC-25 and RC-50 were 101.2%, 96.2%, and 94.0%, respectively, of the experimental axial load. Analytical bending moment of Specimens RC-25, RC-50, and RC-B were 96.2%, 94.0% and 80.3%, respectively, of the experimental bending moment.

![Analytical P-M interaction diagram and experimental results for Group RC using ERSB method.](image)

Analytical P-M interaction diagram of Group MI was constructed using two calculations procedure as explained above. Figure 8.5 presents the analytical and experimental results of P-M interaction diagrams of Group MI. It can be seen from
Figure 8.5 that the first and second calculations results of $P$-$M$ interaction diagram underestimated the corresponding experimental results of $P$-$M$ interaction diagram. Also, the first calculation results of $P$-$M$ interaction diagram at 50 mm eccentric axial load and four-point bending were lower than the corresponding second calculation results of $P$-$M$ interaction diagram. Analytical axial load of Specimens MI-0, MI-25 and MI-50 calculated using the first calculation procedure were 130.3%, 87.1% and 75.3%, respectively, of the experimental axial load. Analytical bending moment of Specimens MI-25, MI-50 and MI-B calculated using the first calculation procedure were 87.1%, 75.3% and 72.1%, respectively, of the experimental bending moment.

Analytical axial load of Specimens MI-0, MI-25 and MI-50 calculated using the second calculation procedure were 130.3%, 88.2% and 79.6%, respectively, of the experimental axial load. Analytical bending moment of Specimens MI-25, MI-50 and MI-B calculated using the second calculation procedure were 88.2%, 79.6% and 89.2%, respectively, of the experimental bending moment.
Figure 8.5. Analytical $P$-$M$ interaction diagram and experimental results for Group MI using ERSB method.

Analytical $P$-$M$ interaction diagram of Group MA was constructed using two calculation procedures. Figure 8.6 presents the analytical and experimental results of $P$-$M$ interaction diagrams of Group MA. It can be seen from Figure 8.6 that the first and the second calculation results of $P$-$M$ interaction diagram underestimated the corresponding experimental results of $P$-$M$ interaction diagram. In addition, the first calculation results of $P$-$M$ interaction diagram at 50 mm eccentric axial load and four-point bending were slightly lower than the corresponding second calculation results of $P$-$M$ interaction diagram. Analytical axial load of Specimens MA-0, MA-25 and MA-50 calculated using the first calculation procedure were 104.3%, 94.8% and 85.1%, respectively, of the experimental axial load. Analytical bending moment of Specimens MA-25, MA-50 and MA-B calculated using the first calculation procedure were 91.7%, 85.1% and 76.0%, respectively, of the experimental bending moment.
Analytical axial load of Specimens MA-0, MA-25 and MA-50 calculated using the second calculation procedure were 104.3%, 95.6% and 88.6%, respectively, of the experimental axial load. Analytical bending moment of Specimens MA-25, MA-50 and MA-B calculated using the second calculation procedure were 92.5%, 88.6% and 88.9%, respectively, of the experimental bending moment.

Figure 8.6. Analytical $P-M$ interaction diagram and experimental results for Group MA using ERSB method.

Analytical $P-M$ interaction diagram of Group HY was constructed using two calculation procedures. Figure 8.7 presents the analytical and experimental results of $P-M$ interaction diagrams of Group HY. It can be seen from Figure 8.7 that the first and the second calculation producer underestimated the corresponding experimental results of $P-M$ interaction diagrams at 25 mm and 50 mm eccentric axial loads and four-point bending. However, the first calculation results of $P-M$ interaction diagram at 50 mm
eccentric axial load and four-point bending were lower than the corresponding the second calculation results of $P-M$ interaction diagram. Analytical axial load of Specimens HY-0, HY-25 and HY-50 calculated using the first calculation procedure were 82.8%, 99.0% and 89.6%, respectively, of the experimental axial load. Analytical bending moment of Specimens HY-25, HY-50 and HY-B calculated using the first calculation procedure were 97.6%, 89.6% and 61.8%, respectively, of the experimental bending moment.

Analytical axial load of Specimens HY-0, HY-25 and HY-50 calculated using the second calculation procedure were 82.8%, 99.9% and 93.9%, respectively, of the experimental axial load. Analytical bending moment of Specimens HY-25, HY-50 and HY-B calculated using the second calculation procedure were 98.5%, 93.9% and 74.4%, respectively, of the experimental bending moment.

![Figure 8.7](image.png)

Figure 8.7. Analytical $P-M$ interaction diagram and experimental results for Group HY using ERSB method.
It was concluded from the results of the first and the second calculation procedure that both calculation results of $P-M$ interaction diagram underestimated the corresponding experimental results of $P-M$ interaction diagram. It was found that the analytical results of $P-M$ interaction diagrams constructed using the model recommended by ACI 544.4R-09 (ACI 2009) result in lower values than the analytical results of $P-M$ interaction diagram constructed using the model proposed by Bentur and Mindess (2006). The difference between the two calculation procedures was observed for the axial load and bending moment values for the specimens tested under 50 mm eccentric axial load and four-point bending.

8.4.3 The L-L Integration Method

Layer by Layer (L-L) integration method was used to draw analytical $P-M$ interaction diagrams. In this method, the cross-section of the specimen was divided into a number of small layers. Figure 8.8 presents stress and strain distribution in the RC circular cross-section area. In each layer on the compression side (above the neutral axis) of the specimen cross-section, two types of concrete including the concrete cover (unconfined) and concrete core (confined) were considered. In order to model the contribution of the concrete cover on the compression side, Equation 8.1 was used.

The compressive strength and corresponding strain of the specimen improve due to lateral confinement provided by helices or ties to the concrete core (consider as a confined concrete). Also, the inclusion of steel fibres can provide indirect confinement pressure to the concrete core by delaying cover spalling and reducing the cracks propagation (Paultrê et al. 2010). Hence, In order to model the contribution of the
concrete core on the compression side, Equation 8.1 was used by replacing the $f'_c$ and $\varepsilon_c'$ with the maximum compressive stress ($f_{cc}'$) and corresponding strain ($\varepsilon_{cc}'$) of confined concrete. The models proposed by Paul tre et al. (2010) to calculate to $f_{cc}'$ and $\varepsilon_{cc}'$ were used in this study and are expressed as follows:

$$f_{cc}' = f'_c \left[ 1 + 2.4 \left( \frac{f'_l}{f'_c} \right)^{0.7} \right]$$  (8.13a)

$$\varepsilon_{cc}' = \varepsilon_c' + 0.21 \left( \frac{f'_l}{f'_c} \right)^{1.7}$$  (8.13b)

where $f'_l$ is the effective confinement pressure provided by lateral reinforcement (helical) and it is equal to $f'_l = \frac{k_e}{2dcs'} \frac{A_{sh}f_{hy}}{\rho_s}$, where $k_e$ is the confinement effectiveness coefficient and it is equal to $k_e = \left[ \frac{[1-s'/2d_c]}{1-\rho_{cc}} \right]$, where $d_c$ is the diameter of concrete core, $s'$ is the spacing between two helices from centre to centre, $\rho_{cc}$ is the ratio of the area of the longitudinal reinforcement to the area of the concrete core, $f_{hy}$ is the yield tensile stress of the steel helix, and $A_{sh}$ is the area of steel helix.

The indirect confinement pressure provided by the steel fibres to the concrete core can be calculated using Equation 8.14:

$$f'_{ls} = \eta_\theta \tau_f (I.R.1 + I.R.2)$$  (8.14)
where, $I.R.1$ refers to the reinforced index of first type of steel fibres, $I.R.2$ refers to the reinforced index of second type of steel fibres. Hence, the effective confinement pressure due to the helices and the steel fibres of the specimens reinforced with steel fibres can be calculated as follows:

$$f_{ts}' = f_t' + f_{ls}' \quad (8.15)$$

In each layer on the tension side (below the neutral axis) of the cross-section SFR-HSC specimen, the concrete was considered as an unconfined concrete. In order to model the contribution of the SFR-HSC on the tension side of the cross-section, tensile stress-strain model proposed in the literature for steel fibre reinforced concrete was used in this study. The tensile stress-strain model proposed by Lok and Xiao (1998) for steel fibre reinforced concrete has been adopted in this study. Lok and Xiao (1998) considered tensile stress-strain curve of steel fibre reinforced concrete constituted by three branches. Figure 8.8 presents a typical tensile stress-strain curve of steel fibre reinforced concrete. An elastic behaviour up to the maximum tensile stress ($f_{ct}$) and corresponding strain ($\varepsilon_{ct}$) were adopted, followed by a linearly decreasing curve to the point of a residual tensile strength ($f_{rt}$) and corresponding strain ($\varepsilon_{rt}$). The elastic modulus of concrete in tension, $E_{ct}$, was considered to be same as elastic modulus of concrete in compression. The tensile stress-strain model of steel fibre reinforced concrete is expressed as follows:
\[ f_t = \begin{cases} -f_{ct} \left[ 2 \left( \frac{\varepsilon_t}{\varepsilon_{ct}} \right) - \left( \frac{\varepsilon_t}{\varepsilon_{ct}} \right)^2 \right] & -\varepsilon_{rt} \leq \varepsilon_t \leq 0 \\ -f_{rt} & \varepsilon_t \leq -\varepsilon_{rt} \end{cases} \]  

(8.16a)

\[ f_{ct} = f_m [1 - v_t] + 0.5 f_{rt} \]  

(8.16b)

\[ \varepsilon_{ct} = 2 \left( \frac{f_{ct}}{E_{ct}} \right) \]  

(8.16c)

\[ f_{rt} = \eta \tau f \sum_{n=1}^{n=2} l. R. \]  

(8.16d)

\[ \varepsilon_{rt} = \tau f \left[ \frac{l_f}{d_f} \right] \left( \frac{1}{E_{sf}} \right) \]  

(8.16e)

where \( f_t \) and \( \varepsilon_t \) are the tensile stress and corresponding strain of steel fibre reinforced concrete, respectively, at any point, \( E_{sf} \) is the modulus of elasticity of the steel fibre, \( f_{rt} \) is the residual tensile stress of steel fibre.
The depth \((d_i)\) of each layer has to be small enough to obtain accurate results (Yazici and Hadi 2009). For each layer, the depth was taken as 1 mm. Also, the average total width \((w_{total})\), the average core width \((w_{core})\), the average cover width \((w_{cover})\) and strain of each layer in circular cross-section can be determined as follows:

\[
w_{total} = 2 \sqrt{\left(\frac{D}{2}\right)^2 - \left(\frac{D}{2} - \left(i - \frac{1}{2}\right)d_i\right)^2} \tag{8.17a}
\]

\[
w_{core} = 2 \sqrt{\left(\frac{d_c}{2}\right)^2 - \left(\frac{D}{2} - \left(i - \frac{1}{2}\right)d_i\right)^2} \quad \text{For} \quad 2 \left(\frac{d_c}{2}\right)^2 - \left(\frac{D}{2} - (i - \frac{1}{2})d_i\right)^2 > 0 \tag{8.17b}
\]
\[ w_{\text{cover}} = w_{\text{total}} - w_{\text{core}} \]  

(8.17c)

where \( i \) is the number of layers, \( D \) is the diameter of the circular cross-section (200 mm), \( d_c \) is the diameter of the core cross-section (150 mm), and \( \varepsilon_{ci} \) is the strain in each layer. The value of \( \varepsilon_{ci} \) is positive for the layers above the neutral axis on the compression side, and it is negative for the layers below the neutral axis on the tension side. The compression forces of each concrete (cover and core) layer in the compression and tension side of the concrete layer in tension side can be determined using Equations 8.19 and 8.20, respectively.

\[ \varepsilon_{ci} = \left[ 1 - \left( i - \frac{1}{2} \right) \frac{d_x}{d_n} \right] \varepsilon_{cu} \]  

(8.18)

\[ F_{c,\text{core}} = f_{ci} w_{\text{core}} d_i \]  

(8.19a)

\[ F_{c,\text{cover}} = f_{ci} w_{\text{cover}} d_i \]  

(8.19b)

\[ F_{ti} = f_{ti} w_{\text{total}} d_i \]  

(8.20)

where \( F_{c,\text{core}} \) is the compression force of each concrete core layer on the compression side. \( F_{c,\text{cover}} \) is compression force of each concrete cover layer on the compression side, \( F_{ti} \) is the tension force of each concrete layer on the tension side. The nominal axial load of the cross-section can be calculated as follows:

\[ P_n = \sum_{i=1}^{n} (F_{c,\text{core}} + F_{c,\text{cover}} + F_{ti}) + \sum_{i=1}^{k} F_{sx} \]  

(8.21)
The bending moment in each layer can be calculated by taking the moment of the force for each layer around the centroid of the cross-section of the specimen. The nominal bending moment of the specimens can be determined as follows:

\[
M_n = \sum_{i=1}^{n} \left( F_{c,\text{core}} + F_{c,\text{cover}} + F_{t_i} \right) \left[ \frac{D}{2} - \left( i - \frac{1}{2} \right) d_i \right] + \sum_{i=1}^{k} F_{sx} \left( \frac{D}{2} - d_x \right)
\]

(8.22)

Based on an iterative procedure, an MS Excel spreadsheet was used to determine the area, stress, strain, axial load and bending moment for each layer, and to draw the \(P-M\) interaction diagrams of the specimens.
Figure 8.9. Distribution of stress and strain in the L-L integration method for the circular cross-section.
8.4.2.1 Analytical \( P-M \) Interaction Diagrams Using the L-L Integration Method

Figure 8.10 shows the \( P-M \) interaction diagram of the experimental and analytical results of Group RC. It can be seen from Figure 8.10 that the experimental \( P-M \) interaction diagram of Group RC fit very well with the analytical results. Analytical axial loads of Specimens RC-0, RC-25 and RC-50 were 101.0%, 96.2%, and 94.0%, respectively, of the experimental axial loads. Analytical bending moments of Specimens RC-25, RC-50, and RC-B were 99.1%, 97.8% and 90.2%, respectively, of the experimental bending moments.

![Analytical P-M Interaction Diagrams Using the L-L Integration Method](image)

Figure 8.10. Analytical \( P-M \) interaction diagram and experimental results for Group RC using L-L integration method.

Analytical \( P-M \) interaction diagram of Group MI was constructed using the L-L integration method. Figure 8.11 presents the analytical and experimental results of \( P-M \)
interaction diagram of Group MI. It can be seen from Figure 8.11 that the analytical results, except Specimen MI-0, underestimated the corresponding experimental $P-M$ interaction diagram. This conservative estimate is because analytical $P-M$ interaction diagram were calculated based on concrete strain ($\varepsilon_{cu}$) in compression and concrete strain ($\varepsilon_{ct}$) in tension. Furthermore, in eccentrically loaded specimens, the actually concrete strain may exceed the $\varepsilon_{cu}$ and $\varepsilon_{ct}$ in most compression and tension regions. Analytical axial loads of Specimens MI-0, MI-25 and MI-50 were 131.9%, 89.5% and 82.9%, respectively, of the experimental axial loads. Analytical bending moments of Specimens MI-25, MI-50 and MI-B were 89.5%, 82.9% and 93.7%, respectively, of the experimental bending moments.

Figure 8.11. Analytical $P-M$ interaction diagram and experimental results for Group MI using L-L integration method.
Analytical $P-M$ interaction diagram of Group MA was constructed using the L-L integration method. Figure 8.12 presents the analytical and experimental results of $P-M$ interaction diagram of Group MA. It can be seen from Figure 8.12 that the analytical results, except analytical result of Specimen MA-0, underestimated the corresponding experimental $P-M$ interaction diagram. Analytical axial loads of Specimens MA-0, MA-25 and MA-50 were 105.2%, 99.5% and 94.6%, respectively, of the experimental axial loads. Analytical bending moments of Specimens MA-25, MA-50 and MA-B were 96.3%, 94.6% and 91.7%, respectively, of the experimental bending moments.

![Graph showing analytical and experimental results for Group MA using L-L integration method.](image)

Figure 8.12. Analytical $P-M$ interaction diagram and experimental results for Group MA using L-L integration method.

Analytical $P-M$ interaction diagram of Group HY was constructed using the L-L integration method. Figure 8.13 presents the analytical and experimental results of $P-M$ interaction diagram of Group HY. It can be seen from Figure 8.13 that the analytical
results underestimated the corresponding experimental $P$-$M$ interaction diagram. Analytical axial loads of Specimens HY-0, HY-25 and HY-50 were 90.0%, 100.8% and 97.5%, respectively, of the experimental axial loads. Analytical bending moments of Specimens HY-25, HY-50 and HY-B were 99.3%, 97.5% and 77.5%, respectively, of the experimental bending moments.

![Graph of Analytical $P$-$M$ Interaction Diagram and Experimental Results for Group HY Using L-L Integration Method](image)

Figure 8.13. Analytical $P$-$M$ interaction diagram and experimental results for Group HY using L-L integration method.

### 8.4.4 The ERSB Method vs the L-L Integration Method

Figure 8.14 shows a comparison between the experimental and two different analytical results for Group RC. It can be seen from Figure 8.14 that the analytical results of $P$-$M$ interaction diagram constructed from the L-L integration method are close to the experimental results compared to the analytical results constructed from ERSB method. This could be because the distribution of the stress in the L-L integration method is non-linear. However, a uniform stress distribution is considered in the ERSB method.
Figure 8.14. Analytical $P$-$M$ interaction diagram and experimental results for Group RC using ERSB method and L-L integration method.

Figure 8.15 shows a comparison between the experimental and two different analytical results for Groups MI, MA and HY. In general, it can be seen from Figure 8.15 that the analytical results of $P$-$M$ interaction diagram constructed from the L-L integration method were higher than the analytical results constructed from the ERSB method. Also, the analytical results from the L-L integration and ERSB methods underestimated the corresponding experimental results. Thus, it can be concluded that the use of stress-strain models proposed in the literature in the L-L integration method can accurately predict the axial load and corresponding bending moment values of the specimens reinforced with different types of steel fibres. Also, it was found that the use of ERSB method is easier than the use of the L-L method to determine the $P$-$M$ interaction diagrams of the specimens.
Figure 8.15. Analytical $P$-$M$ interaction diagram and experimental results for Groups (a) MI,
(b) MA and (c) HY using the ERSB method and the L-L integration method.
8.5  Moment-Curvature Response

In this study, the analytical bending moment-curvature \((M-\phi)\) curves of the specimens tested under 25 mm and 50 mm eccentric axial loads were developed using the L-L integration method. The strain distribution along the cross-section and strain for each steel bar can be calculated using Equation 8.18 and 8.3, respectively. The force and the moment of each steel bar were determined using Equations 8.4 and 8.5, respectively. The forces of the concrete in compression side and tension side were calculated using Equations 8.19 and 8.20, respectively.

For the specimens tested under initial eccentricity, the cross-section is supposed to carry bending moment due to the application of the initial eccentricity at the end of the specimen and lateral deformation due to the curvature along the height of the specimens as shown in Figure 8.16. Thus, the bending moment at mid-height of the specimens \((M_m)\) can be calculated as follows:

\[
M_m = P(e_i + \Delta_m)
\]  

(8.23)

where \(P\) is the axial load and \(\Delta_m\) is the lateral deformation at mid-height of the specimen, \(e_i\) is the initial eccentricity. To calculate \(\Delta_m\) at the mid-height of the specimens, the deformed shape can be assumed to be a half-sine curve as described in previous research study (Bazant et al. 1991). Thus, the \(\Delta_m\) can be determined as follows:
\[ \Delta_m = \frac{H^2}{\pi^2} \phi_m \]  
\[ \phi_m = \frac{\varepsilon_{cu}}{d_n} \]

where \( H \) is the height of the specimen and \( \phi_m \) is the curvature at mid-height of the specimen.

Figure 8.16. Typical deformation of pin-ended specimens tested under eccentric axial load.

Figure 8.17 presents the flowchart of analytical development of the \( M-\phi \) curve. The following steps present the summary of the flowchart:
1. Input the longitudinal bar properties: the distance between the centre of the longitudinal bars to the extreme compression fibre in the compression side \((d_c)\), yield tensile strain \((\varepsilon_y)\), the total cross-section area of the bar \((A_s)\), modulus of elasticity of the bar. Also, input material properties of concrete: the compressive strength of unconfined SFR-HSC \((f'_{ct})\), the axial strain corresponding \((\varepsilon'_{ct})\) to \(f'_{ct}\), material parameter \((\beta_1)\), modulus of elasticity of SFR-HSC, the compressive strength \((f'_{cc})\) and corresponding strain \((\varepsilon'_{cc})\) of confined SFR-HSC, the maximum tensile stress \((f_{ct})\) and corresponding strain \((\varepsilon_{ct})\) of SFR-HSC, modulus of elasticity of SFR-HSC in tension, \(E_{ct}\), residual tensile stress \((f_{rt})\) and corresponding strain \((\varepsilon_{rt})\) of concrete in tension.

2. Select an initial value for concrete strain \((\varepsilon_c)\) at the extreme concrete fibre in the compression side. The initial value for \(\varepsilon_c\) was about 0.0005.

3. Select an initial value for the depth of the neutral axis \((d_n)\).

4. Calculate the total width of the concrete layer \((w_{total})\), the width of the concrete core \((w_{core})\), the width of the concrete cover \((w_{cover})\), strain for each concrete layer \((\varepsilon_{ci})\) and force \((F_{c,core}, F_{c,cover})\) for each concrete layer in the compression side and the force \((F_{ci})\) for each concrete layer on the tension side. In addition, calculate the strain \((\varepsilon_{st})\) and the force \((F_{st})\) for each longitudinal bar. Also, determine the axial load \((P)\) and bending moment \((M)\), lateral deformation \((\Delta_m)\) and curvature \((\phi)\).
5. Calculate the error from Equation 8.25. If the error from the previous steps is greater than 2%, then repeat Steps 3 to 5 by an increment in the depth of neutral axis \((d_n)\) with the addition of concrete layer \((i)\) until the error becomes lower than 2%. The error can be calculated as follows

\[
Error = \left| \frac{e_i + \Delta_m - M/P}{e_i + \Delta_m} \right| \times 100
\]  

(8.25)

6. Repeat Steps 2 to 5 with an increment of 0.0005 in \(\varepsilon_c\) value until \(\varepsilon_c\) value is equal to \(\varepsilon'_c\) value.

The above calculations and the analytical procedures were implemented by developing a computer program using MATLAB (2013b). The MATLAB code is presented in Appendix A.
Figure 8.17. A flowchart of analytical development of $M$-$\phi$ diagram.
8.5.1 Analytical $M$-$\phi$ Response

Figure 8.18 shows the experimental and analytical of $M$-$\phi$ curves of the specimens tested under 25 mm and 50 mm eccentric axial loads. The experimental $M$-$\phi$ curves of the specimens tested under 25 mm and 50 mm eccentric axial loads were calculated and presented in Chapter 7. In general, it can be seen from Figure 8.18a that a reasonable agreement can be found between the experimental and analytical results of $M$-$\phi$ curves for specimens tested under 25 mm eccentric axial load.

Figure 8.18b shows that the experimental results of $M$-$\phi$ curves of specimens tested under 50 mm eccentric axial load underestimated of the analytical results $M$-$\phi$ curves. It was observed that the use of the L-L integration method to determine the $M$-$\phi$ curves can accurately predict the experimental results of $M$-$\phi$ curves.
Figure 8.18. Compression between analytical and experimental $M-\phi$ responses of the specimens tested under (a) 25 mm and (b) 50 mm eccentric axial loads.
8.6 Parametric Study

A parametric study was conducted to investigate the effects of different parameters of micro steel fibres such as volume content \( (v_f \%) \) of steel fibre and aspect ratio \( (l_f/d_f) \) of steel fibres on the \( P-M \) interaction diagram of SFR-HSC circular specimens. Group MI was employed as a reference for the parametric study. The effect of volume content and aspect ratio of steel fibres on the \( P-M \) interaction diagram was investigated using the L-L integration method.

8.6.1 Volume Content of Steel Fibre

The volume content of steel fibres ranged from 1\% to 4\%. Figure 8.19 presents the effect of different volume content \( (v_f\%) \) on the \( P-M \) interaction diagram. It was found that an increase in volume content of steel fibres results in a slight reduction in the axial load for specimens tested under different eccentric axial load. However, for the specimens tested under four-point bending, an increase in volume content of steel fibres leads to increase the bending moment capacity.
8.6.2 Aspect Ratio of Steel Fibre

Figure 8.20 shows the effect of the different aspect ratio of steel fibres on the $P$-$M$ interaction diagram. The aspect ratio of steel fibres ranged from 30 to 60. It can be seen from Figure 8.20 that the axial load and bending moment of the specimens tested under different eccentric axial load were not affected with an increase in the aspect ratio of the steel fibres. Whereas an increase of the aspect ratio of steel fibres was more pronounced for the bending moment capacity of the specimens tested under four-point bending.
8.6.3 Reinforced Index of Steel Fibre

Reinforced Index (R.I.) is the interaction between volume content and aspect ratio of steel fibres. The R.I. of the steel fibres ranged from 0.3 to 1.8. Figure 8.21 shows the effect of the different R.I. of steel fibres on the P-M interaction diagram. It can be seen from Figure 8.21 that an increase in R.I. of steel fibres results in a slight reduction in the axial load for specimens tested under different eccentric axial load. In addition, an increase of the R.I. of steel fibres leads to increase the bending moment capacity of the specimens.
In this chapter the analytical results of the preliminary testing program and the main experimental program were presented. The analytical results of the preliminary testing program were presented to discuss the analytical stress-strain behaviour of HSC and SFR-HSC cylinders. The analytical results of the main experimental program were presented to discuss the analytical $P$-$M$ interaction diagrams using two methods (Equivalent rectangular stress block and Layer by Layer integration methods), and the analytical $M$-$\phi$ response of the specimens tested under 25 mm and 50 mm eccentric axial loads. In addition, a parametric study was conducted to investigate the influence of volume content and aspect ratio of steel fibres on the $P$-$M$ interaction diagrams. Furthermore, codes in MATLAB were developed to determine $M$-$\phi$ response of the SFR-HSC specimens tested under 25 mm and 50 mm eccentric axial loads.

Figure 8.21. Effect of the $R.I.$ of steel fibre on the $P$-$M$ interaction diagrams of SFR-HSC specimens.

### 8.7 Summary

In this chapter the analytical results of the preliminary testing program and the main experimental program were presented. The analytical results of the preliminary testing program were presented to discuss the analytical stress-strain behaviour of HSC and SFR-HSC cylinders. The analytical results of the main experimental program were presented to discuss the analytical $P$-$M$ interaction diagrams using two methods (Equivalent rectangular stress block and Layer by Layer integration methods), and the analytical $M$-$\phi$ response of the specimens tested under 25 mm and 50 mm eccentric axial loads. In addition, a parametric study was conducted to investigate the influence of volume content and aspect ratio of steel fibres on the $P$-$M$ interaction diagrams. Furthermore, codes in MATLAB were developed to determine $M$-$\phi$ response of the SFR-HSC specimens tested under 25 mm and 50 mm eccentric axial loads.
A comparison between the analytical and experimental results was carried out. In general, it can be observed from the analytical calculation have a good agreement with experimental results. Conclusions and recommendations that can be drawn from the analytical and experimental studies are presented in the next chapter.
9: CONCLUSIONS AND RECOMMENDATIONS

9.1 Conclusions

Analytical investigations were conducted to propose strength models using Artificial Neural Network (ANN) analysis. Based on the available research studies, two sets of databases were compiled and analysed. The first set of the databases was an experimental investigation database of 102 Steel Fibre Reinforced Concrete (SFRC) cylinders, which were used to propose ANN compressive strength model and ANN splitting tensile strength model. The ANN compressive strength and ANN splitting tensile strength models were developed as functions of the compressive strength of concrete, the volume content of steel fibre, the aspect ratio of steel fibre and the nominal tensile strength of the steel fibres. The second set of database was an experimental investigation database of 71 Steel Fibre Reinforced Normal Strength Concrete (SFR-NSC) and Steel Fibre Reinforced High Strength Concrete (SFR-HSC) square columns, which were used to propose ANN Purelin and ANN Tansig compressive strength models of SFR-NSC and SFR-HSC square columns. The ANN Purelin and ANN Tansig compressive strength models were developed as a function of the compressive strength of concrete, confinement pressure provided by lateral reinforcement and confinement pressure provided by steel fibres.

A pilot study was conducted to investigate the behaviour of the High Strength Concrete (HSC) with and without different types of steel fibres (micro steel fibres, macro steel fibres and hybrid steel fibres). A total of 40 cylinders of HSC with and without steel fibres, divided into four groups were cast and tested. The influence of different
parameters, including volume content and aspect ratio on the compressive strength and splitting tensile strength of the HSC were investigated.

Experimental investigations were conducted to determine the behaviour of HSC columns with and without steel fibres under different loading conditions (concentric axial load, 25 mm and 50 mm eccentric axial loads, and four-point bending). A total of sixteen circular concrete columns, divided into four groups were cast and tested. The influence of different parameters, including types, volume content and aspect ratio of steel fibre (micro steel fibre, macro steel fibre and hybrid steel fibres) and loading condition (concentric axial load, 25 mm and 50 mm eccentric axial loads, and four-point bending), on the behaviour of concrete columns were investigated.

Analytical investigations were also conducted to construct the compressive stress-strain curves of HSC and SFR-HSC cylinders. Also, the analytical axial load-bending moment interaction diagrams of HSC with and without steel fibres specimens were constructed using two analytical methods, i.e., Equivalent Rectangular Stress Block (ERSB) and Layer-by-Layer (L-L) integration methods. The L-L integration method was used to analyse the cross-section of SFR-HSC columns tested under 25 mm and 50 mm eccentric axial load and construct the analytical bending moment-curvature curves. Furthermore, a parametric study was conducted to investigate the influence of different volume content and aspect ratio of micro steel fibres on the axial load-bending moment interaction diagrams of SFR-HSC specimens.

Based on the analytical and experimental investigations conducted in the research study, the following conclusions are drawn:
1. The proposed ANN compressive strength model and ANN splitting tensile strength model can accurately predict the experimental results of SFRC. It was found that the coloration coefficients ($R^2$) between the predict results and experimental results were 0.96 and 0.86 of the ANN compressive strength model and ANN splitting tensile strength model, respectively. Furthermore, the MAE of the ANN compressive strength model and ANN splitting tensile strength model was less than 10% and 20%, respectively.

2. The proposed ANN Purelin compressive strength model and ANN Tansig compressive strength model of SFR-NSC and SFR-HSC square columns can accurately predict the experimental results. Although the ANN Tansig compressive strength model is more complicate than the ANN Purelin compressive strength model, the coloration coefficients ($R^2$) of the ANN Tansig compressive strength model is higher than the $R^2$ of the ANN Purelin compressive strength model. The $R^2$ between the prediction results and experimental results of SFR-NSC and SFR-HSC square columns was 0.72 and 0.93 of the ANN Purelin and ANN Tansig compressive strength models, respectively. In addition, the MAE of the ANN Purelin and ANN Tansig compressive strength models was less than 10% and 20%, respectively.

3. The compressive strength of the HSC was marginally influenced by the inclusion of different types of steel fibre. The higher improvement in compressive strength of the HSC was observed with the inclusion of 3%, 2% and 2.5% of micro, macro and hybrid steel fibres, respectively. Furthermore, it was found that the inclusion of 3%, 2%, and 2.5% by volume of micro, macro, and hybrid steel fibres, respectively, into
the HSC produced a reasonable improvement in splitting tensile strength of the HSC with a small reduction in the workability of the fresh HSC mixture.

4. The compressive strength of SFR-HSC decreased with an increase of the aspect ratio of steel fibres. It was observed that the use of 2% by volume of micro steel fibres and macro steel fibres, an increase in the aspect ratio of the fibres results in reducing the average compressive strength by 60%. However, the use of 3% by volume of micro steel fibres and macro steel fibres, an increase in aspect ratio of the fibres results in decreasing the average compressive strength by 80%.

5. For Specimens RC-0, MI-0, MA-0 and HY-0 tested under concentric axial load, the inclusion of the steel fibres leads to delaying the spalling of concrete cover. Specimen HY-0 exhibited higher axial load than Specimens MI-0, MA-0 and RC-0. Specimen MI-0 carried lower axial load than Specimen RC-0. The first peak axial load ($P_{1p}$) of the specimens was compared with the nominal axial load ($N_{no}$). It was observed that the ratio of $P_{1p}/N_{no}$ was 0.77, 0.96 and 1.21 for Specimens MI-0, MA-0 and HY-0, respectively.

6. The ductility of the specimens tested under concentric axial load increased with the inclusion of the steel fibres. Specimen HY-0 exhibited higher ductility than Specimens MI-0, MA-0 and RC-0. The ductility of Specimens MI-0, MA-0 and HY-0 were increased by 19%, 16% and 27%, respectively, compared to Specimen RC-0.

7. For Specimens RC-25, MI-25, MA-25 and HY-25 tested under 25 mm eccentric axial load, higher improvement in the maximum axial load was observed for Specimen MI-25. The maximum axial load of Specimens MI-25, MA-25 and HY-25
was increased by 28%, 9% and 11%, respectively, compared to Specimen RC-25. Specimen HY-25 exhibited higher improvement in the ductility. The ductility of Specimens MI-25, MA-25 and HY-25 was increased by 20%, 35% and 53%, respectively, compared to Specimen RC-25.

8. For Specimens RC-50, MI-50, MA-50 and HY-50 tested under 50 mm eccentric axial load, Specimen MI-50 exhibited higher improvement in the maximum axial load. The maximum axial load of Specimens MI-50, MA-50 and HY-50 was increased by 46%, 22% and 23%, respectively, compared to Specimen RC-50. The higher improvement in ductility was observed for Specimen HY-50. The enhancement in ductility was 33%, 63% and 65% for Specimens MI-50, MA-50 and HY-50, respectively, compared to Specimen RC-50.

9. For Specimens RC-B, MI-B, MA-B and HY-B tested under four-point bending (four-point load), the highest improvement in the maximum bending load was found for Specimen HY-B. The maximum bending load increased by 17%, 11% and 35% for Specimens MI-B, MA-B and HY-B, respectively, compared to Specimen RC-B. The ductility of the specimens tested under four-point bending was significantly improved by the addition of steel fibres. Specimen MA-B showed higher improvement in the ductility. The ductility increased by 12%, 75% and 11% for Specimens MI-B, MA-B and HY-B, respectively, compared to Specimen RC-B.

10. The experimental axial load-bending moment ($P$-$M$) interaction diagrams of Group MI, MA and HY were higher than $P$-$M$ interaction diagram of Group RC.
11. The experimental bending moment-curvature (M-ϕ) diagram of Specimen MI-25 was higher than M-ϕ diagrams of Specimens MA-25 and HY-25. In addition, the M-ϕ diagram of Specimen MI-50 was higher than M-ϕ diagrams of Specimens MA-50 and HY-50.

12. Increasing the load eccentricity leads to a reduction in the maximum axial load and the ductility of the specimens.


15. The analytical P-M interaction diagrams of SFR-HSC specimens constructed using the L-L integration method was closer to the experimental P-M interaction diagrams than the P-M interaction diagrams constructed using the ERSB method.
16. The analytical $M-\phi$ curves of Specimens MI-25, MA-25, HY-25, MI-50, MA-50 and HY-50 constructed using L-L integration method were closer to the experimental $M-\phi$ curves.

17. The parametric study showed that an increase volume content of steel fibres results in a slight reduction in the axial load for specimens tested under different eccentric axial load. However, for the specimens tested under four-point bending, an increase in volume content of steel fibres leads to increase the bending moment capacity.

18. The parametric study also showed that the axial load and bending moment of the specimens tested under different eccentric axial load were not affected by an increase in the aspect ratio of the steel fibres. Whereas an increase of the aspect ratio of steel fibres was more pronounced for the bending moment capacity of the specimens tested under four-point bending.

9.2 Recommendations for Future Studies

From the investigations of SFR-HSC specimens, the following future research studies can be recommended:

1. The same research studies can be conducted to investigate the behaviour of different types of concrete such as self-compact concrete, geopolymer concrete and reactive powder concrete specimens with and without steel fibres.
2. The behaviour of SFR-HSC specimens under dynamic load can be conducted to investigate the seismic behaviour of SFR-HSC specimen.

3. The research study can be conducted to investigate the effect of the columns height (slenderness effect).

4. The research study can be conducted to determine the size effect of the different cylinder’s specimens.
REFERENCES

ACI (American Concrete Institute) (2005), "Building Code Requirements for Structural Concrete." *ACI 318-05*, Farmington Hills, MI, USA.

ACI (American Concrete Institute) (2014), "Building Code Requirements for Structural Concrete and Commentary." *ACI 318-14*, Farmington Hills, MI, USA.


ACI (American Concrete Institute) (2010), "Guide to Quality Control and Testing of High-Strength Concrete." *ACI 363R-10*, Farmington Hills, MI, USA.

ACI (American Concrete Institute) (2009), "Design Considerations for Steel Fiber Reinforced Concrete." *ACI 544.4R-09*, Farmington Hills, MI, USA.


CSA (Canadian Standards Association), (2004), "Design of Concrete Structures," *CSA A23.3-04*, Ontario, Canada.


Kovler, K., Sikuler, J. and Bentur, A., (1992), "Free and restrained shrinkage of fibre reinforced concrete with low polypropylene fibre content at early age." *In Fibre*


MATLAB. (2013), MathWorks MATLAB R2013b.


Standards Australia, (2014), "Methods of testing concrete-Determination of properties related to the consistency of concrete - Slump test" AS 1012.3.1-14, Sydney, Australia.


APPENDIX A: MATLAB CODES

A.1: MATLAB CODE FOR ANALYTICAL BENDING MOMENT-CURVATURE CURVES OF SPECIMENS MI-25.

clc; clear all; close all;
d=[169, 138.25, 138.25, 66.75, 66.75, 36];
fco=62;
E1=8470*fco^0.33;
eco=0.0028;
r=(E1/(E1-(fco/eco)));
B1=4.18;
fcco=100.;
ec=0.0144;
r2=(E1/(E1-(fcco/ec)));
fct=-fco*((1-0.03)+((0.5*0.4*0.9*(0.6*(fco^0.66)))));
Ect=E1;
ect=-2*fct/Ect;
es1=-0.5*0.9*0.6*(fco^0.66);
fret=30*0.6*(fco^0.66)*((1/200000));
ey=0.0023;
As=113.04;
Es=200000;
ei=25;
l=0;
for ecu=0:0.0005: ecc;
l=l+1;
go=1;
for c=40:1:200;
if go==l;
i=0;
for N=1:1:c;
i=i+1;
bt=2*(102.5^2-(102.5-(N-0.5))^2)^(0.5);
ec(i)=(c-(N-0.5))*ecu/c;
if N<=20;
bc(i)=0;
bu(i)=bt;
fcc(i)=0;
fco(i)=0;
if ec(i)<=eco && ec(i)>0;
fco(i)=fco*r*(ec(i)/eco)/((r-1)+(ec(i)/eco)^r);
elseif ec(i)>eco;
fco(i)=fco*B1*(ec(i)/eco)/((B1-1)+(ec(i)/eco)^B1);
else fco(i)=0;
end
Pc(i)=(bu(i)*fco(i)+bc(i)*fcc(i))/1000;
Mc(i)=Pc(i)*(102.5-(N-0.5))/1000;
end
if N>20 && N<=c;
bc(i)=2*(82.5^2-(102.5-(N-0.5))^2)^(0.5);
bu(i)=bt-bc(i);
fcc(i)=(r2*fcco*(ec(i)/ecc))/((r2-1)+(ec(i)/ecc)^r2);
if ec(i)<=eco && ec(i)>0;
fcco(i)=fco*r*(ec(i)/eco)/((r-1)+(ec(i)/eco)^r);
elseif ec(i)>eco;
fcco(i)=fco*B1*(ec(i)/eco)/((B1-1)+(ec(i)/eco)^B1);
else fcco(i)=0;
end
Pc(i)=(bu(i)*fcco(i)+bc(i)*fcc(i))/1000;
Mc(i)=Pc(i)*(102.5-(N-0.5))/1000;
end
if N>c;
bc(i)=0;
bu(i)=bt;
fcc(i)=0;
if ec(i)>ect && ec(i)<0;
    fcot(i)=fct*(2*(ec(i)/es1)-(ec(i)/es1));
elseif ec(i) > es1 && ec(i) < ect;
    fcot(i)=ec(i)*0.5*200000*0.03;
elseif ec(i)<es1;
    fcot(i)=fret;
end
Pc(i)=(bu(i)*fcot(i))/1000;
Mc(i)=Pc(i)*(102.5-(N-0.5))/1000;
end
end
for j=1:1:6;
es(j)=(c-d(j))*ecu/c;
if abs (es(j)) <= ey;
    Ps(j)=es(j)*Es*As/1000;
elseif abs(es(j)) > ey;
    Ps(j)=(abs(es(j))/es(j))*ey*Es*As/1000;
end
Ms(j)=Ps(j)*(102.5-d(j))/1000;
end
P=sum(Pc)+sum(Ps);
M=sum(Mc)+sum(Ms);
e=1000*M/P;
k=ecu/c;
def=k*(800/pi)^2;
error=100*abs(ei+def-e)/(ei+def);
if error<2;
go=88;
PP(l)=P;
MM(l)=M;
k1(l)=k*10^6;
end

clc; clear all; close all;
d=[169, 138.25, 138.25, 66.75, 66.75, 36];
fco= 48.38;
E1=8470*fco^0.33;
eco=0.0034;
r=(E1/(E1-(fco/eco)));
B1=6.37;
fcco=86.55;
ecc=0.016;
r2=(E1/(E1-(fcco/ecc)));
fct=-0.7*fco^0.5;
Ect=E1/2;
ect=-fct/Ect;
es1=-0.0023*2;
fret=3.8;
ey=0.0023;
As=113.04;
Es=200000;

for ecu=0:0.0005:ecc;
    l=0;
    for c=40:1:200;
        plot(k1,MM);
if go==1;
i=0;
for N=1:1:c;
i=i+1;
b\(i=2\left(102.5^2-(102.5-(N-0.5))^2\right)^{0.5};\)
ce\(c(i)=(c-(N-0.5))^2/c;\)
if N<=40;
bc\(i=0;\)
bu\(i=bt;\)
fcc\(i=0;\)
f\(cot\(i=0;\)
if ec\(i<=ec_0 && ec\(i>0;\)
f\(c(i)=f_{co}\cdot r_1\cdot (ec\(i)/ec_0)/(r_1+(ec\(i)/ec_0)^r_1);\)
elseif ec\(i>ec_0;\)
f\(c(i)= f_{co}\cdot B_1\cdot (ec\(i)/ec_0)/(B_1-1+(ec\(i)/ec_0)^{B_1});\)
else fc\(i=0;\)
end
Pc\(i=(bu\(i)*fc\(i)+bc\(i)*fcc\(i))/1000;\)
Mc\(i= Pc\(i*(102.5-(N-0.5))/1000;\)
end
if N>40 && N<=c;
bc\(i=2\left(82.5^2-(102.5-(N-0.5))^2\right)^{0.5};\)
bu\(i=bt-bc\(i;\)
fcc\(i=(r_2\cdot f_{cco}\cdot (ec\(i)/ecc))/(r_2-1+(ec\(i)/ecc)^{r_2});\)
if ec\(i<=ec_0 && ec\(i>0;\)
f\(c(i)=f_{co}\cdot r_1\cdot (ec\(i)/ec_0)/(r-1+(ec\(i)/ec_0)^{r});\)
elseif ec\(i>ec_0;\)
f\(c(i)= f_{co}\cdot B_1\cdot (ec\(i)/ec_0)/(B_1-1+(ec\(i)/ec_0)^{B_1});\)
else fc\(i=0;\)
end
Pc\(i=(bu\(i)*fc\(i)+bc\(i)*fcc\(i))/1000;\)
Mc\(i= Pc\(i*(102.5-(N-0.5))/1000;\)
end
if N > c;
bc(i)=0;
bu(i)=bt;
fcc(i)=0;
if ec(i)>ect && ec(i)<0;
ficot(i)=ec(i)*Ect;
elseif ec(i) > es1 && ec(i) < ect;
ficot(i)=fret+((fcot-fret)*(ec(i)+es1))/(ect-es1);
elseif ec(i)<es1;
ficot(i)=fret;
end
Pc(i)=(bu(i)*fcot(i))/1000;
Mc(i)=Pc(i)*(102.5-(N-0.5))/1000;
end
end
for j=1:1:6;
es(j)=(c-d(j))*ecu/c;
if abs (es(j)) <= ey;
Ps(j)=es(j)*Es*As/1000;
elseif abs(es(j)) > ey;
Ps(j)=(abs(es(j))/es(j))*ey*Es*As/1000;
end
Ms(j)=Ps(j)*(102.5-d(j))/1000;
end
P=sum(Pc)+sum(Ps);
M=sum(Mc)+sum(Ms);
e=1000*M/P;
k=ecu/c;
def=k*(800/pi)^2;
error=100*abs(ei+def-e)/(ei+def);
if error<2;
go=88;
PP(l)=P;
MM(l)=M;
k1(l)=k\times10^6;
end
end
end
plot(k1,MM);
## APPENDIX B: READING OF STRAIN GAUGES

### B.1: READING OF STRAIN GAUGES OF STEEL REINFORCEMENT.

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen</th>
<th>Strain gauges on longitudinal reinforcement (mm/mm)</th>
<th>Strain gauges on helical reinforcement (mm/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Compression side</td>
<td>Tension side</td>
</tr>
<tr>
<td>RC</td>
<td>RC-0</td>
<td>0.0022</td>
<td>0.0024</td>
</tr>
<tr>
<td></td>
<td>RC-25</td>
<td>0.002</td>
<td>0.0021</td>
</tr>
<tr>
<td></td>
<td>RC-50</td>
<td>0.002</td>
<td>0.0022</td>
</tr>
<tr>
<td></td>
<td>RC-B</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>MI</td>
<td>MI-0</td>
<td>0.0024</td>
<td>0.0026</td>
</tr>
<tr>
<td></td>
<td>MI-25</td>
<td>0.0029</td>
<td>0.0031</td>
</tr>
<tr>
<td></td>
<td>MI-50</td>
<td>0.002</td>
<td>0.002</td>
</tr>
<tr>
<td></td>
<td>MI-B</td>
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<td>-</td>
</tr>
<tr>
<td>MA</td>
<td>MA-0</td>
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<td>0.003</td>
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<td>-</td>
</tr>
<tr>
<td></td>
<td>MA-50</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>MA-B</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
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<td>0.004</td>
</tr>
<tr>
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<td>0.0029</td>
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<td>0.003</td>
<td>0.002</td>
</tr>
<tr>
<td></td>
<td>HY-B</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>