Methods of using modified wire mesh composite to strengthen concrete with different strengths

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METHODS OF USING MODIFIED WIRE MESH COMPOSITE TO STRENGTHEN CONCRETE WITH DIFFERENT STRENGTHS

HUA ZHAO

This thesis is presented as part of the requirements for the conferral of the degree:

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DECLARATION

I, Hua Zhao, declare that this thesis submitted in partial fulfilment of the requirements for the award of Doctor of Philosophy, from the University of Wollongong, is wholly my own work unless referenced or acknowledged. This document has not been submitted for qualifications at any other academic institution.

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ABSTRACT

Strengthening of existing concrete infrastructure has become increasingly important within the construction and building industry throughout the world. The commonly applied methods for strengthening concrete structures are conventional steel reinforcement, steel plates, fibre reinforced polymers (FRP) jackets, steel wire mesh and mortar composite. Using conventional steel reinforcement to strengthen the existing structure leads to a large change in geometry. The FRP jacketing is a widely applied technology for repairing concrete members. However, both FRP jackets and steel plates require epoxy resins. Steel wire mesh and mortar, also considered as ferrocement, is a kind of high performance composite material due to its tensile strength-to-weight ratio and the ability to undergo large deformation (ACI549R 1997). However, wire mesh is commonly applied with low strength mortar or normal strength concrete (NSC). It is well known that the brittleness of concrete will increase with the increase of concrete strength. The need to investigate the potential to extend wire mesh to high strength cementitious matrices and integrate it with different reinforcement materials has arisen.

To address the research gap on applying wire mesh with high strength cementitious matrices, three series of experiments were conducted. In the first experiment, wire mesh and high strength mortar was used to strengthen high strength concrete (HSC) through the wrapping method; in the second experiment, medium strength concrete (MSC) was cast with wire mesh; in the third experiment, wire mesh and modified high strength mortar (MHSM) were used to strengthen NSC. In addition, the composite of wire mesh and MHSM combined with steel hoops at a range of reinforcement volumetric ratios was used to strengthen NSC. The influence of the number of mesh layers (fraction ratio, $V_f$), steel reinforcement volumetric ratio ($\rho_s$), matrix strength and cover thickness were investigated. Cylindrical specimens and square 12.7×12.7 mm galvanised wire mesh (E12.7WM) were used for the three experiments, considering mechanical properties and economic constraints. No longitudinal steel reinforcement was applied.

The main findings through the comprehensive investigation are as follows:
Unlike HSC cover spalling, which occurred in a sudden manner and was relatively easy to identify, the MHSM cover was unloaded gradually due to the presence of wire mesh. With the increase of number of wire mesh (fracture ratio, $V_f$), the cover spalled more. Moreover, wire mesh exhibited good energy absorption capacity.

For strengthening HSC, the increment in the load carrying capacity tended to increase with the increase of mortar shell area. The test results suggest that the ultimate load was mainly sustained by matrices. Although the amount of reinforcement applied cannot provide the minimum effective confining pressure required by AS3600 (2009), it was sufficient to change the sudden and brittle failure mode attributed to HSC. The ductility improved moderately at a low fraction ratio.

For cast MSC with wire mesh tube reinforcement, the load carrying capacity increased marginally, while ductility increased significantly. The effect of wire mesh was pronouncedly on ductility than on the load carrying capacity. The increment in the layer of wire mesh did not ensure the increase in the ultimate load, which is contrast to what commonly reported in the literature. For the specimens externally confined with wire mesh and no cover applied, the load carrying capacity increased significantly and ductility improved modestly.

For strengthening NSC with steel, wire mesh and MHSM composite, compared to the specimens confined with MHSM only, the ultimate load did not improve for nearly all the specimens strengthened either with wire mesh or steel hoops and wire mesh. However, there is clear and significant improvement in ductility. The specimens confined with relatively low steel volumetric ratio ($\rho_s = 0.97\%, \rho_s = 1.45\%$) with three layers of wire mesh ($\rho_w = 0.53\%$) achieved ductility levels comparable to those well-confined concrete, evidenced by a desirable long load plateau or a second peak load. Specimens confined with only three layers of wire mesh may potentially perform comparably to those with low steel volumetric ratio. The yield stress of transverse steel reinforcement is generally used for determining the confining pressure. However, this study confirms that lateral steel confinement did not reach the yield stress at the second peak load despite NSC core. The proposed equations provide estimations for different cases with good agreement.
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NOTATION

$A_c$ = total area of concrete core, in mm$^2$

$A_g$ = gross cross-sectional area of a compression member, in mm$^2$

$A_s$ = cross-sectional area of transverse steel reinforcement, in mm$^2$

$A_w$ = cross-sectional area of single wire, in mm$^2$

$AUC$ = area under the axial load and axial displacement curves, in kN.mm

$D$ = diameter of confined or reinforced specimen, in mm

$D_c$ = diameter of concrete core, in mm

$D_m$ = overall dimension of the mean of the wire mesh layers, in mm

$D_s$ = overall dimension measured between centre-lines of the steel hoops; in mm

$D_o$ = overall dimension measured between the outermost wires, in mm

$d_s$ = diameter of steel hoop, in mm

$d_w$ = diameter of single wire, in mm

$E_c$ = modulus of elasticity of concrete, in MPa

$f'_c$ = characteristic compressive (cylinder) strength of concrete at 28 days, in MPa

$f_{co}$ = in-situ compression strength of unconfined concrete, in MPa

$f'_{co}$ = compressive strength of unconfined column, in MPa

$f_{cc}$ = compression strength of confined concrete, in MPa

$f_{w}^{l}$ = confining pressure attributed to wire mesh, in MPa

$f_{mp}^{l}$ = confining pressure based on mesh encapsulated mortar plates, in MPa

$f_{sl}^{l}$ = confining pressure contributed by steel hoops, in MPa

$f_{m}^{l}$ = compressive strength of mortar at 28 days, in MPa

$f_{r}$ = the average confining pressure on the core at the level of lateral reinforcement, in MPa

$f_{r, eff}$ = effective confining pressure, in MPa

$f_{ys}$ = yield stress of transverse steel reinforcement, in MPa

$f_{s}$ = stress of steel hoop, in MPa

$f_{uw}^{l}$ = ultimate stress of lateral wire, in MPa

$f_{yw}^{l}$ = yield stress of lateral wire, in MPa

$H$ = specimen height, in mm
$h$ = thickness of ferrocement section, in mm

$k_i$ = confinement effectiveness coefficient

$ke$ = the effectiveness factor accounting for arrangement of steel hoops

$kw$ = the effectiveness factor accounting for arrangement of wire mesh

$n$ = number of mesh layers

$P_y$ = yield load, in kN

$P_u$ = ultimate load, in kN

$P_1$ = ultimate strength of member in compression without taking into the
effect of transvers confinement, in kN

$P_2$ = ultimate strength of member in compression taking into account for the
effect of transvers confinement, in kN

$P_{mod}$ = predicted load, in kN

$P_{sf}$ = load at first spiral fracture, in kN

$s$ = spacing of steel hoops, in mm

$S_{max}$ = maximum spacing of the transverse reinforcement, in mm

$S_r$ = specific surface of wire mesh reinforcement

$s_w$ = lateral wire spacing, in mm

$T$ = the test results of the ultimate tension load of ferrocement plate, in kN

$t$ = protective cover, in mm

$t_{js}$ = equivalent thickness of steel tube due to steel hoops, in mm

$t_{tw}$ = equivalent thickness of steel tube due to wire mesh, in mm

$V_f$ = wire mesh volume fraction ratio, with respect to the mortar shell

$V_{ft}$ = transverse wire volume fraction ratio, with respect to the mortar shell

$\Delta_{85}$ = the post-yield axial displacement at 85% of the ultimate load, in mm

$\Delta_{fr}$ = the axial displacement at first hoop fracture, in mm

$\Delta_u$ = the axial displacement at the ultimate load, in mm

$\Delta_y$ = the axial displacement at the yield load, in mm

$\varepsilon$ = axial strain

$\varepsilon_{cc}$ = axial strain of the confined concrete corresponding to the compressive
strength of the confined concrete

$\varepsilon_{cr}$ = cracking strain of concrete in direct tension

$\lambda_{ts}$ = equivalent steel tube confinement ratio due to transverse steel

$\lambda_{tw}$ = equivalent steel tube confinement ratio due to transverse wire
\( \mu_{ts} \) = equivalent steel tube reinforcement ratio due to transverse steel  
\( \mu_{tw} \) = equivalent steel tube reinforcement ratio due to transverse wire  
\( \mu_y \) = ductility, computed as \( \Delta_u/\Delta_y \)  
\( \mu_{85} \) = ductility, computed as \( \Delta_{85}/\Delta_y \)  
\( \mu_{sf} \) = ductility, computed as \( \Delta_{sf}/\Delta_y \)  
\( \nu \) = Poisson’s ratio  
\( \rho \) = concrete density, in kg/m³  
\( \rho_s \) = transverse steel reinforcement volumetric ratio  
\( \rho_w \) = transverse wire volumetric ratio  
\( \rho_{s+M} \) = volumetric ratio of transverse steel and wire, with varying number of mesh layers
CHAPTER 1 INTRODUCTION

1.1 Background

Strengthening of existing concrete infrastructure has become increasingly important within the construction and building industry throughout the world. The commonly applied methods for strengthening concrete structures are conventional steel reinforcement, steel plates, fibre reinforced polymers (FRP) jackets, steel wire mesh and mortar composite. Steel wire mesh and mortar composite is also recognised as a kind of ferrocement. Using conventional steel reinforcement to strengthen the existing structure leads to a large change in geometry. FRP jacketing is a widely applied technology for repairing concrete members (Triantafillou et al. 2012). However, both steel plates and FRP jackets require epoxy resins.

Steel wire mesh and mortar composite, also considered as ferrocement, which is a kind of high performance composite material due to its tensile strength-to-weight ratio and the ability to undergo large deformation. Studies have suggested that the ferrocement jacket can significantly improve both the load carrying capacity and ductility of concrete columns (Balaguru 1988; Waliuddin and Rafieeqi 1994; Rao and Seshu 1998; Xiong et al. 2011). As a different means of strengthening and repairing concrete structures, ferrocement can be applied at a very low material cost and a low level of technical skills. This study is focused on welded galvanised steel wire mesh (from heron ‘wire mesh’), which is commonly applied with low strength mortar or normal strength concrete (NSC).

However, the need to investigate the potential to extend wire mesh to high strength cementitious matrix and integrate it with different reinforcement materials has arisen. To address the research gap on applying wire mesh with high strength cementitious matrices, this thesis presents a series of experiments on applying wire mesh to strengthen or reinforce concrete with a range of strengths. In this study the effect of wire mesh was investigated with a range of variables. Considering mechanical properties and economic constraints, cylindrical specimens and welded galvanised steel wire mesh 12.7×12.7 mm (S12.7WM) were adopted for the three experiments.
1.2 Aims

This thesis aims to: investigate the behaviour of concrete confined with two types of composites under concentric loading, namely wire mesh and high strength mortar, and steel hoops, wire mesh and modified high strength mortar (MHSM); examine the effect of wire mesh on structural behaviors of strengthened concrete; and develop expressions for predicting the ultimate load and the strength of wire mesh composite confined concrete with satisfactory accuracy.

1.3 Objectives

Complying with the aims mentioned above, this study has been conducted with the following specific objectives:

(1) To investigate the mechanical properties of the composite and the constituent materials;

(2) To develop a method of using wire mesh with high strength matrix to strengthen concrete member and improve the performance of mortar matrices as a very economical material;

(3) To examine the effect of the wire mesh and high strength mortar confinement for HSC and MSC;

(4) To investigate the effect of using wire mesh only as external confinement (with no cover);

(5) To examine the effect of the steel hoops, wire mesh and MHSM composite for NSC under concentric loading;

(6) To investigate the combination of low steel volumetric ratios and different wire mesh fraction ratios;

(7) To investigate the influence of cover thickness and strength on the structural behaviour;

(8) To provide expressions for predicting the strength of wire mesh composite confined concrete and evaluate the methods.
1.4 Scope of the Thesis

This research includes that: 1) investigation of the properties of the confinement composite and constituent materials of the composite; and 2) the structural behaviour of confined concrete with a range of compressive strengths under concentric loading.

1.5 Thesis Outline

This thesis comprises eight chapters. The first chapter introduces the background and the focus of this study. The next chapter presents a literature review on the existing models of steel confined concrete, wire mesh confined concrete, and the current development. Chapter 3 provides the detailed exploration of applying wire mesh to high strength concrete (HSC) and medium strength concrete (MSC). The trial test on using steel reinforcement, wire mesh and MHSM to strength normal strength concrete (NSC) is presented in Chapter 4. Chapter 5 presents the detailed design of the third experimental work. Chapter 6 shows the preliminary tests conducted on the composite and the constituent materials of the composite. Chapter 7 contains the third experiment test results and the detailed discussion. Finally, conclusions are drawn in Chapter 8 as well as recommendations for further research.
CHAPTER 2  LITERATURE REVIEW

2.1 Introduction

In this chapter a concise overview of various confinement materials is presented first, followed by the characteristics and development of using wire mesh as confinement. As mentioned in Chapter 1, the improved composite consists both steel reinforcement and wire mesh. The development on the existing compressive strength models for steel confined concrete is reviewed chronologically; a review on wire mesh reinforced concrete is presented, with an emphasis on the strength of confined concrete and the ultimate load of strengthened members. This study is focused on confined concrete with circular cross section.

2.2 Confinement Materials for Strengthening Concrete

2.2.1 Different Types of Confinement Materials

Strengthening of existing concrete infrastructure has become increasingly important within the construction and building industry throughout the world. The commonly applied methods for strengthening concrete structures are conventional steel reinforcement, steel plates, fibre reinforced polymers (FRP) jackets, steel wire mesh and mortar composite. Steel wire mesh and mortar composite is also recognised as ferrocement. Using conventional steel reinforcement to strengthen the existing structure leads to a large change in geometry. FRP jacketing is a widely applied technology for repairing concrete members due to its excellent properties such as extremely high strength-to-weight ratio and corrosion resistance (Triantafillou et al. 2006). Numerous showed that FRP jackets can considerably improve the strength and ductility of RC columns (Hadi 2007a; Hadi 2007b; Lam and Teng 2002). For heavily FRP confined cylinders, approximately bilinear stress-strain behaviour is exhibited, indicating a very brittle failure and no postpeak behaviour. Both steel plates and FRP jackets require epoxy resins. The disadvantages of using epoxy as bonding agent include: 1) hazards associated with handling the eczema and toxic components; 2) low permeability and diffusion tightness, provoking freeze and thaw problems; 3) poor thermal compatibility, leading unfavourable constraints; 4)
requirements on the dry surface of base concrete and the temperature at application, usually under 10°C; 5) emission impact on environment (Blanksvärd et al. 2009; Al-Abdwais and Al-Mahaidi 2017); and 6) potential de-bonding between concrete and strengthening materials such as FRP sheets and steel plates (Engindeniz et al. 2005).

Other studies have been conducted on different types of mesh (Bentayeb et al. 2008; Choi 2008; Hadi and Zhao 2011; Wang et al. 2015). Wire mesh is commonly applied to low strength mortar or normal strength concrete. There is a trend of applying wire mesh with relatively high strength concrete or cementitious matrix to construct or strengthen concrete members (Kumar and Rao 2006; Mourad and Shannag 2012). The third experiment of this study is focused on the composite of steel reinforcement, wire mesh, and modified high strength mortar.

2.2.2 Characteristics of Ferrocement

Steel wire mesh and mortar composite is also considered as a kind of ferrocement. Ferrocement is a form of reinforced concrete that differs from conventional reinforced concrete mainly by the manner in which reinforcing wire mesh distributed. Normally, Ferrocement is constructed from hydraulic cement mortar reinforced with closely spaced layers of relatively small-diameter continuous mesh (ACI 549R-1997). As there are many types of wire mesh in terms of the fabrication, geometry and materials of mesh. This study is only focus on used square welded steel wire mesh with an aperture of 12.7×12.7 mm and a diameter of 1 mm, approximately.

Ferrocement behaves significantly different from conventional reinforced concrete in terms of strength (Kondraivendhan and Pradhan 2009). Ferrocement is considered as a homogenous material, having homogeneous properties in two dimensions. It presents high tensile strength, high modulus of rupture and excellent cracking performance. Additionally, high specific surfaces of wires enhance the development of bond forces with the cementitious matrix. As a result, smaller crack spacing and width appear (Naaman 2000).

Currently, there is no standard for ferrocement. According to ACI 549R (1997) and ACI 549.1R (1993), the main characteristics of ferrocement include: (1) high tensile...
strength–to-weight ratio, which facilitates structure durability through superior cracking behaviour such as smallness of crack widths and a large number of cracking, compared to reinforced concrete; (2) light weight, which is very important in seismic zones as inertia forces are essential in those areas; (3) not flammable, compared to alternative materials with similar thickness such as steel and fibre glass, ferrocement might be a preferable choice; (4) forming formwork, which makes ferrocement especially suitable for repairs of structures with curved surfaces; (5) good permeability; (6) low material cost and easy availability; and (7) environmentally-friendly compared to FRP. In addition, ferrocement demonstrates good impact resistance, high toughness and excellent capacity of deformation.

2.2.3 Development of Ferrocement

Ferrocement is primarily applied to shell and folded plate elements. However, it is also adopted as compressive load bearing elements. In this case ferrocement is usually applied in three ways: 1) casting concrete with wire mesh; 2) precasting ferrocement shell; and 3) wrapping wire mesh and packing it with mortar. The first method can be used as general enhancement for the load carrying capacity and ductility for a new structural member. The second method can be used in fair-face permanent form, providing additional strength and ductility. The third method is commonly used for preparing and strengthening of existing structural members (Waliuddin and Rafeeqi 1994).

In addition to the investigation of the behaviour of wire mesh confined concrete members under axial compression (Balaguru 1988; Kaushik and Singh 1997; Kondraivendhan and Pradhan 2009), research has been carried on using circular ferrocement jackets to strengthen square reinforced concrete (RC) columns subjected to cyclic lateral forces and constant axial load (Takiguchi and Abdullah 2001). In their study cement slurry with compressive strength of 30 MPa was adopted and the tested results showed that the displacement ductility of the confined columns was improved significantly, while the original columns illustrated shear failure at low displacement ductility.
Ferrocement jacket is also applied to strengthen preloaded (to failure or up to 67% or 85% of the ultimate load) square concrete columns (Fahmy et al. 1999). The test results showed that the repaired columns with ferrocement jacket exhibited considerable increment in axial load carrying capacity and stiffness. Moreover, ferrocement is used to strengthen masonry columns. The ferrocement reinforced brick column demonstrated a substantial improvement in terms of the load carrying capacity, ductility, serviceability and cracking patterns (Shah 2011). Recently, it is approved that micro-mesh reinforcement and high strength slurry can be effectively applied for repairing, rehabilitation and strengthening of existing structural members such as floors, columns and walls (Flohrer and Tschotschel 2012). Refer to Fig. 2.1 for the visualised information.

![Fig. 2.1 Micro-mesh reinforcement (Flohrer and Tschotschel 2012)](image)

As mentioned earlier, there is a trend to extend the application of wire mesh to high strength cementitious matrices. Shannag and Mourad (2012) investigated mortar with relatively high strength and flowability for ferrocement. It was suggested that high strength mortar and wire mesh composite exhibits preferable improvement in the load carrying capacity and ductility. For specimens confined with the jacket containing two or four layers of wire mesh, the axial stress increased 16% and 29%, respectively; the axial strain increased 32% and 70%, respectively.

On the other hand, there is some limitation associated with ferrocement. In the case of pure tension, the load carrying capacity increases with direct proportion to the amount of reinforcement (Lodi et al. 2010). However, in the case of pure
compression, the effect of the vertical wires on the load carrying capacity of the member is limited. Furthermore, the risk of corrosion is potentially high due to high surface area of wire reinforcement (ACI 549R-1997). It is noted that corrosion is not included in the work scope of this study.

2.3 Characteristics of Concrete under Compression

2.3.1 Concrete Strength Classification

According to ACI 363R (1992), concrete with design compressive strength of 41 MPa or greater is considered as HSC. It is also recognised that the definition of high-strength concrete is geographically different. Depending on the availability of the compressive strength of commercially produced concrete, the recognition could be varied from 83 MPa to 103 MPa. In Australia, concrete with compressive strength ranged from 50 MPa to 100 MPa is considered as high strength (AS3600-2009). Normal strength concrete typically has a compressive strength of between 20 MPa to 40 MPa (Kosmatka 2008).

2.3.2 Passive Confinement

Back dated to 1899, the advantages associated with spiral reinforcement were initially pointed by Considere (Martinez et al. 1984). Richart et al. (1928, 1929) conducted intensive experimental work on confined concrete and indicated that compared to the concrete with closely confined spiral, the strength of concrete with active confinement from lateral pressure of fluid was approximately the same. At low levels of compressive stress in the concrete, the expansion of core is limited. Accordingly, the transverse reinforcement is hardly stressed. As concrete stress approaching the uniaxial strength, the dilatation of concrete core increases significantly due to progressive internal cracking while the transverse reinforcement restrains the deformation. Through the interaction between the core and the confinement, confining pressure developed. Therefore, in conventionally reinforce and externally jacketed concrete columns, confining pressure is passive in nature. Passive confinement may be constant or variable through an axial load history. Constant confining pressure is generated if the confining material behaves in plastic
manner, for example, transverse reinforcing steel yielding. By contrast, FRP jackets and steel that are still elastic generate variable confining pressure (Harries and Kharel 2002). It is well known that the stress-strain relationship of concrete can be significantly enhanced at large strains by transverse reinforcement; while the effect on strength is much smaller. Additionally, transverse steel can prevent lateral buckling of the longitudinal reinforcing steel and act as shear reinforcement (Park and Paulay 1975; Samra 1990).

According to Mirmiran and Shahaway (1997), two conditions need to be satisfied in a confinement mechanism. The first is about the strain compatibility between the core and the confined member, while the second is regarding the force equilibrium in the confine section. As the confining pressure is a function of lateral strain, which is depending on Poisson’s ratio, the two conditions are interdependent. Refer to Fig. 2.2 and Fig. 2.3 for the confining pressure on the core and steel reinforcement.

Fig. 2.2 Confining pressure
2.3.3 Volume Dilation of Concrete

The composite nature of concrete is mainly attributed to volume dilation. As pointed by Shah and Chandra (1968), the cement paste itself does not expand under the compression load. To the contrary, the paste specimen continues to consolidate up to failure. Volumetric expansion is observed only when the cement paste is mixed up with aggregates. As appointed by Chen and Han (2007), the stress at which volume starts increasing is associated with a clear increase of microcracks through mortar matrix. Moreover, the mechanical behaviour of concrete is influenced by the development of micro cracking (Hsu et al. 1963). The relationship between axial stress and volume strain under biaxial compression is shown in Fig. 2.4. After the first stage in compression, concrete eventually dilates because of micro cracking with the increase in compression (Newman and Newman 1969).
2.4 Models for the Compressive Strength of Steel Confined Concrete

Generally, design-oriented or analysis-oriented models are applied for predicting the confined concrete strength. The difference is that: the former is mainly based on interpretation or curve-fitting of test results; while the latter is on the basis of incremental numerical procedures, in view of equilibrium and compatibility (Li 2005). The following section provides an overview of the models for steel confined circular concrete columns.

Through an extensive experimental program, Richart, Brandtzaeg, and Brown (1928, 1929), established the model for the confined strength and corresponding strain for spirally reinforced and hydraulically confined columns, shown as Eq. 2.1. This equation can be presented in a conventional way as Eq. 2.2, while the corresponding strain is shown as Eq. 2.3.

\[
f'_{cc} = f'_{co} + 4.1f_t \tag{2.1}
\]

\[
\frac{f'_{cc}}{f_{co}} = 1 + k \frac{f_t}{f_{co}} \tag{2.2}
\]
where $f'_{cc}$ and $\varepsilon_{cc}$ = compressive strength and the corresponding strain of the confined concrete, respectively; $f_{co}$ and $\varepsilon_{co}$ = the unconfined concrete strength and the corresponding strain, respectively; $f_l$ = lateral confining pressure, calculated through force equilibrium; $k_1$ and $k_2$ = confinement effectiveness coefficient, $k_1=4.1$ and $k_2=5k_1$, respectively. It is noted that $f_{co}$ is based on the compressive strength of standard cylinder.

Balmer 1949 conducted the triaxial compression test on standard cylinders Ø152 × 305 mm. The specimens were all cast from the same mix, fog-cured for either 28 or 90 days. The confining pressure was applied from 7 MPa to 172 MPa. The results indicated that the increase in strength with confinement was nonlinear. The compressive strength of the confined specimens can be presented as Eq. 2.4. It is noted the notation is as defined previously. Since then, the investigation has been continuously carried on and extended to transverse reinforcement with various arrangements and different cross sections.

\[
\frac{f'_{cc}}{f_{co}} = \left(1 + 9.175 \frac{f_l}{f_{co}} \right)^{0.73}
\]  

(2.4)

Popovics (1973) proposed Eq. 2.5 to describe the relationship between axial stress and axial strain of unconfined normal weight concrete up to medium strength. Three parameters, $f_o$, $\varepsilon_o$, and $E$ were adopted to present the unconfined concrete stress-strain diagram. This stress and strain equation is widely employed for the analysis-oriented models (Xiao et al. 2010).

\[
f = E\varepsilon \frac{n-1}{n-1+(\varepsilon/\varepsilon_o)^n}
\]  

(2.5)

where $f$ = axial stress; $E$ = initial modulus of elasticity; $\varepsilon$ = unit strain in concrete; $\varepsilon_o$ = strain responding to the compressive strength. At $\varepsilon = \varepsilon_o$, $E = (f_{co}/\varepsilon_o)n/(n-1)$, the stress and strain relationship can be presented as follows:
where \( n_{\text{concrete}} = 0.4 \times 10^{-3} f_{co} + 1.0 \); \( n_{\text{mixture}} = 0.15 \times 10^{-3} f_{co} + 1.5 \); \( n_{paste} = 12 \); \( f_{co} \) = ultimate cylinder compressive strength.

Iyengar et al. (1978) conducted on steel spiral confined concrete cylinders with a compressive strength of 34 MPa and suggested Eq. 2.2 can be modified to:

\[
\frac{f}{f_{co}} = \frac{n(e/e_u)}{(n-1) + (e/e_u)^s} \quad (2.6)
\]

where the original value of \( k_1 \) was replaced with 4.6; \( s = \) pitch of spiral, \( d_s = \) diameter of spiral; \( 1 - s/d_s \) = an effective factor accounting for the arrangement of the transverse reinforcement (spiral).

Sheikh and Uzumeri (1980) investigated strength and ductility of steel tie confined square columns. They pointed out a transition period occurred where protective cover started unloading. This concept is illustrated in Fig. 2.5, where the concrete contribution curves are non-dimensionalised through dividing the recorded load by the theoretical force \( P_{occ} \) attributed by concrete of the gross cross section and the theoretical force \( P_{occ} \) due to the contribution of concrete core, respectively. A transition took place from the lower curve to the upper curve, where the cover is partially effective. The load was sustained by the core after concrete cover completely spalled.
It was recommended that the transition started at a strain value $\varepsilon_0$, which corresponds to the maximum plain concrete stress and ends at a strain value $\varepsilon_{50u}$, which corresponds to 50 percent of the peak stress of plain concrete on the descending branch of its stress-strain curve. Based on the standard cylinder tests, the two strain values $\varepsilon_0$ and $\varepsilon_{50u}$ were recommended as 0.002 and 0.0035, respectively (Sheikh and Toklucu 1993). This phenomenon indicates that the gain in the strength of core due to confinement could be overestimate if the ultimate load occurred at a stage while the cover concrete was still partially effective in carrying load.

The concept of effectively confined area was further discussed (Sheikh and Uzumeri 1982). For square section, at the tie level, the effectively confined area was reduced; between the tie spacing, the effectively confined area is further reduced. This concept was reflected as confinement effectiveness coefficient $k_e$. For the case of the effectively confined area of circular lateral reinforcement, the reduction of the core area only occurs along the longitudinal axis of the member (Foster et al. 1998), as shown in Fig. 2.6.
Ahmad and Shah (1982) investigated spirally reinforced concrete cylinders in axial compression. The compressive strength of concrete was up to 69 MPa and both normal strength and high strength steel were adopted in their test. Practically there was no cover applied. It is noted that the axial load and axial displacement curve is different from the case if cover would be applied. The average stress of confined specimens at the peak was presented as Eq. 2.8:

\[ f_{cc} = f_{co} + k_1 (f'_r)_p \]  \hspace{1cm} (2.8)

\[ (f'_r)_p = \frac{\rho_s f_{sw}}{2} \left( 1 - \sqrt{\frac{s}{1.25D_c}} \right) \]  \hspace{1cm} (2.9)

where \( f_{co}, f_{cc} \) are the average stress at the peak of the stress and strain curve of unconfined and confined specimens, respectively; \( k_1 = 6.61(f_o)^{0.5}(f'_r)_p^{0.04}; \) \( \rho_s = 2\pi d_s/(D_c s); \) \( d_s \) = diameter of spiral; \( D_c \) = diameter of the concrete core. Eq. 2.9 indicates that when the pitch of spiral is 1.25 times greater than the confined core, the effective of confinement is negligible (Binici 2005).

Ahmad and Shah (1982b) proposed Eq. 2.10 for the constitutive relationship of plain concrete subjected to an active confining pressure, using an iterative procedure.

\[ y = \frac{A_{x_i} + (D_i - 1)x_i}{1 + (A_i - 2)x_i + D_i x_i^2} \]  \hspace{1cm} (2.10)
where \( y = f_i / f_{cc} \), \( x_i = \varepsilon_i / \varepsilon_{ip} \), \( f_i = \) the most principal compressive stress; \( f_p = \) the most principal compressive strength; \( \varepsilon_i = \) the strain in the \( i \)-th principal direction (\( i = 1, 2 \) or \( 3 \)); \( \varepsilon_{ip} = \) the strain at the peak in the \( i \)-th direction; \( A_i = E_i / E_{ip} \), \( E_i \) is the initial slope of the \( f_i - \varepsilon_i \) curve; \( E_{ip} = f_p / \varepsilon_{ip} \); \( D_i = \) parameter, governing the descending part of the \( f_i - \varepsilon_i \) curve; \( D_1 = 1.111 + 0.876 A_1 - 4.0883 (\tau_{oct} / f_{co}) \); \( D_2 = 3.30 + 0.156 A_1 - 4.466 (\tau_{oct} / f_{co}) \).

The compressive strength \( f_p \) was determined by a strength criterion based on the octahedral theory and experimental data from Eq. 2.11.

\[
\frac{\tau_{oct}}{f_{co}} = A + B \frac{f_{oct}}{f_{co}} \tag{2.11}
\]

where \( f_{oct} = 1/3 (f_p + 2f_r) \), \( \tau_{oct} = (2/3)^{0.5} (f_p - f_r) \), \( A = 0.2261 \) and \( B = 0.7360 \) when \( \tau_{oct} / f_{co} < 1.75 \).

Martinez et al. (1984) investigated spirally reinforced high strength normal and lightweight concrete columns. The concrete cylinder compressive strength ranged from 21 MPa to 69 MPa and 17 MPa to 59 MPa, respectively, for normal weight concrete and lightweight concrete. Eq. 2.12 is recommended for the confined strength of normal weight concrete. It is noted that \( f'_{co} \) is the compressive strength of unconfined column.

\[
\frac{f'_{co}}{f''_{co}} = 1 + 4.0 \left( 1 - \frac{s}{d'} \right) \frac{f_i}{f''_{co}} \quad 21 \leq f'_{c} \leq 83 \text{ MPa} \tag{2.12}
\]

Fafitis and Shah (1985) proposed a stress-strain model for normal weight concrete with compressive strength ranged from 21 MPa to 69 MPa, based on which the confined concrete strength can be presented as Eq. 2.13 by neglecting the parameter \( \lambda_s \) and converting \( \varepsilon_{ip} \) to MPa. \( \lambda_s \) is dependent on the relative confinement and when the relative confinement is small, this parameter is close to 1.

\[
\frac{f_{co}}{f_{co}} = 1 + \left[ 1.15 + \frac{21}{f_{co}} \right] \frac{f_i}{f_{co}} \tag{2.13}
\]
Mander et al. (1988b) developed a general model for concrete subjected to uniaxial compressive loading and confined with different sections (circular or rectangular) under either static or dynamic axial compressive loading. This model is most-cited (Teng et al. 2015). The stress and strain model is illustrated in Fig. 2.7. The curve continues until the first hoop fracture. According to Scott et al. (1982), the point, where the first fracture of a hoop occurred, is considered as the limit of concrete compressive strain.

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

(2.14)

where \( f'_{co} \) = compressive strength of unconfined column; \( f'_c \) = effective confining stress. Li et al. (2001) concluded that the effective confining stress based on the yield
strength of the confining steel leads to overestimation for concrete confined with high yield strength steel. Eq. 2.14 was modified as follows (Li et al. 2001):

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

(2.15)

when \( f_{co} \leq 52 \text{ MPa} \), \( \alpha = (21.2 - 0.35 f_{co}) (f' / f_{co}) \); when \( f_{co} > 52 \text{ MPa} \), \( \alpha = 3.1 (f' / f_{co}) \).

Base on the stress-strain equation proposed by Popovics (1973) as mentioned earlier, the longitudinal compressive concrete stress \( f_c \) is given as following:

\[
\frac{f_c}{f'_{cc}} = \frac{xr}{r - 1 + x'}
\]

(2.16)

\[
x = \frac{\varepsilon_c}{\varepsilon_{co}}
\]

(2.17)

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

(2.18)

where \( \varepsilon_c \) = longitudinal compressive concrete strain; \( \varepsilon_{co} \) and \( \varepsilon_{cc} \) = corresponding strain at \( f_{co} \) and \( f_{cc} \), respectively; the constant \( r \) is defined by Eq. 2.19.

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

(2.19)

where \( E_c = 5000( f'_{co} ) \) MPa, \( E_{sec} = f'_{cc} / \varepsilon_{cc} \). It should be appointed that the widely adopted Mander et al. (1988 a) model lacks of necessary control over the slope of the postpeak branch (Karthik and Mander 2011). In seismic design, predicting the descending branch is very important to ensure proper behaviour of columns, which undergo large deformations.

Saatcioglu and Razvi (1992) proposed an analytical model for constructing the stress-strain relationship of confined concrete. This model comprises a parabolic
ascending branch and a followed-up linear descending branch, beyond which a constant residual strength is assumed at 20% strength level, as shown in Fig. 2.8. The equivalent uniform pressure, which was based on the average lateral pressure, was used to determine the confined concrete strength and the corresponding strain. This model can be applied for columns with different cross section and transverse reinforcement arrangement, shown as Eq. 2.20-Eq. 2.22:

$$\frac{f'_{c}}{f'_{co}} = 1 + k_1 \frac{f_{lc}}{f_{co}}$$  \hspace{1cm} (2.20)

$$\varepsilon_{ec} = \varepsilon_{co} \left(1 + 5K \right)$$  \hspace{1cm} (2.21)

$$\varepsilon_{es} = 260 \rho \varepsilon_{co} + \varepsilon_{o85}$$  \hspace{1cm} (2.22)

where $f_{lc} = k_2 f_{c}$, $k_1 = 6.7(f_{lc})^{0.17}$, $k_2$ was recommended as 1.0 for circular columns confined by spiral or square columns with longitudinal reinforcement and closely spaced transverse ties; $\varepsilon_{co} =$ corresponding to the peak stress of unconfined concrete; $\varepsilon_{ec} =$ corresponding to the peak stress of confined concrete; $K = k_2 f_{lc} / f_{co}$; $\varepsilon_{o85} =$ strain at 85% strength level beyond the peak stress of unconfined concrete; $\varepsilon_{es} =$ strain at 85% strength level beyond the peak stress of confined concrete; $\rho =$
transverse reinforcement volume ratio. Saatcioglu and Razvi (1992) suggested that for lateral pressure, the value of \( k_1 \) is low, while in the high pressure range, the value of \( k_1 \) approximates a constant value. The confined strength for spirally confined circular columns is shown as Eq. 2.23.

\[
\frac{f_{cc}}{f'_{co}} = 1 + 6.7 \frac{f_{le}}{f'_{co}}^{0.83}
\]

(2.23)

The prediction for the parabolic ascending portion is expressed as Eq. 2.24

\[
f_c = f_{cc} \left[ 2 \left( \frac{e_c}{e_{cc}} \right) - \left( \frac{e_c}{e_{cc}} \right)^2 \right] \leq f_{cc}
\]

(2.24)

where \( f_c \) and \( e_c \) = longitudinal compressive concrete stress and the corresponding strain, respectively. It is noted although this analytical model has the advantage of simplicity of application and considering residual strength, a critical point was not specified: the longitudinal compressive strain of concrete at the first hoop fracture.

Li et al. (2001) investigated steel confined concrete columns with circular and square sections. The concrete used had a range of strength from 35.2 MPa to 82.5 MPa and was confined with various amount of lateral reinforcement. The yield strength of transverse reinforcement was 445 MPa and 1318 MPa, respectively. Li et al. (2001) also adopted the five-parameters multiaxial failure criteria of William and Warnke (1975) to describe the theoretical ultimate strength surface. Based on the test results of Khaloo and Ahmad (1989), the strength of confined high strength concrete was proposed as:

\[
f_{cc} = f'_{co} \left[ -0.413 + 1.413 \sqrt{1 + 11.4 \frac{f_i}{f'_{co}} - 2 \frac{f_i}{f'_{co}}} \right]
\]

(2.25)

where \( f'_{co} \) = in-place unconfined concrete compressive strength; the remaining notation is as defined previously. It was suggested for HSC confined with circular normal yield strength confining reinforcement, the confined concrete strength can be approximately presented as Eq. 2.26. For the design purpose, \( f_i \) is the effective confining pressure.
For concrete confined with ultra-high yield strength steel reinforcement, Li et al. (2001) suggested that Eq. 2.27 and Eq. 2.28 can be applied for circular confinement for normal strength and high strength concrete, respectively:

\[
\frac{f_{cc}'}{f_{co}'} = 1 + 4.0 \frac{f'_i}{f'_{co}} \quad \text{NSC} \quad (2.26)
\]

\[
\frac{f_{cc}'}{f_{co}'} = 1 + 4.6 \frac{f'_i}{f'_{co}} \quad \text{NSC} \quad (2.27)
\]

\[
\frac{f_{cc}'}{f_{co}'} = 1 + 2.7 \frac{f'_i}{f'_{co}} \quad \text{HSC} \quad (2.28)
\]

In addition, empirical formulas for the axial strain at the maximum strength (\(\varepsilon_{cc}\)) and the maximum concrete strain (\(\varepsilon_{cu}\)) were proposed. For circular cross section and ordinary strength steel, the axial strain, \(\varepsilon_{cc}\), at the maximum strength is:

\[
\varepsilon_{cc} = 1.0 + 384 \left( \frac{f'}{f_{co}} \right)^2 
\]

(2.29)

For circular cross section with ultra-high strength steel, the axial strain, \(\varepsilon_{cc}\), at the maximum strength is presented in Eq. 2.30 and Eq. 2.31 for concrete with different compressive strength.

\[
\frac{\varepsilon_{cc}}{\varepsilon_{co}} = 1.0 + \left( 120 - 1.554 \frac{f'}{f_{co}} \right) \left[ \frac{f'}{f_{co}} \right] \quad \text{when } f_{co} \leq 50 \text{ MPa} 
\]

(2.30)

\[
\frac{\varepsilon_{cc}}{\varepsilon_{co}} = 1.0 + \left( 71.4 - 0.623 \frac{f'}{f_{co}} \right) \left[ \frac{f'}{f_{co}} \right] \quad \text{when } f_{co} > 50 \text{ MPa} 
\]

(2.31)
The maximum concrete strain, \( \varepsilon_{cu} \), for circular confinement with normal strength steel is proposed as following:

\[
\frac{\varepsilon_{cu}}{\varepsilon_{co}} = 2 + \left( 143.5 - 1.48 f'_{co} \right) \sqrt{\frac{f'}{f_{co}}} \quad \text{when} \quad f'_{co} < 80 \text{ MPa} \tag{2.32}
\]

\[
\frac{\varepsilon_{cu}}{\varepsilon_{co}} = 2 + \left( 89.8 - 0.74 f'_{co} \right) \sqrt{\frac{f'}{f_{co}}} \quad \text{when} \quad f'_{co} \geq 80 \text{ MPa} \tag{2.33}
\]

The maximum concrete strain, \( \varepsilon_{cu} \), for circular confinement with ultra-high strength steel are presented as follows:

\[
\frac{\varepsilon_{cu}}{\varepsilon_{co}} = 2.0 + \left( 86.7 - 1.06 f'_{co} \right) \sqrt{\frac{f'}{f_{co}}} \quad \text{when} \quad f'_{co} \leq 50 \text{ MPa} \tag{2.34}
\]

\[
\frac{\varepsilon_{cu}}{\varepsilon_{co}} = 2.0 + \left( 53.5 - 0.42 f'_{co} \right) \sqrt{\frac{f'}{f_{co}}} \quad \text{when} \quad f'_{co} > 50 \text{ MPa} \tag{2.35}
\]

According to AS3600 (2009), for the transverse steel reinforcement confined concrete, the effective confining pressure can be computed using Eq. 2.36.

\[
f_{r, \text{eff}} = \left( 1 - \frac{s}{2D_s} \right)^2 \left( \frac{2A_y f_{ys}}{D_s s} \right)
\]

where \((1-s/2D_s)^2\) is a simplified effectiveness factor accounting for the arrangement of the lateral reinforcement, noted as \(k_c\); \((2A_y f_{ys}/D_s s)\) is the average confining pressure on the core cross-section taken at the level of lateral reinforcement, noted as \(f'_c\); \(A_y = \) cross-sectional area of one leg of the fitment; \(f_{ys} = \) yield stress of the confinement; \(D_s = \) overall dimension measured between centre-lines of the outermost reinforcement; \(s = \) centre to centre spacing of lateral reinforcement along the specimen.
Eid et al. (2010) proposed a theoretical model on the stress-strain relationship of circular concrete columns with steel spirals or hoops. The analytical curve was derived from elastoplastic analysis.

The complete stress-strain curve of the confined core is mainly described through the parameters: the confined concrete strength $f_{cc}$, the corresponding strain $\varepsilon_{cc}$, the post peak strain at 85% of the confined concrete strength $\varepsilon_{85}$, the concrete residual strength $f_{cr}$ and the ultimate concrete strain $\varepsilon_{cu}$.

The prepeak branch ($\varepsilon_c \leq \varepsilon_{cc}$) is expressed (Hoshikuma et al. 1997):

$$f_c = E_c \varepsilon_c \left(1 - \frac{1}{n_a} \left(\frac{\varepsilon_c}{\varepsilon_{cc}}\right)^{\alpha-1}\right) \quad (2.37)$$

where $n_a = \frac{E_c \varepsilon_{cc}}{E_c \varepsilon_{cc} - f_{cc}}$; $E_c = \text{concrete modulus of elasticity of unconfined concrete}$; $f_{cc} = \text{confined concrete strengthen}$; $\varepsilon_{cc} = \text{corresponding strain to } f_{cc}$.. $f_{cc}$ can be determined:

$$\frac{f_{cc}}{f_{cr}} = 0.47 \rho_v - 0.021 + \sqrt{1.043 + 4.97 \rho_v} \quad (2.38)$$

where $\rho_v = \rho_v \left(f_{op} / f_{cr}\right)$; $\rho_v = \phi_v \pi / \left(2a(s + \phi_i)\right)$; $a = \text{cylinder radius}$; $\phi_i = \text{cross-section diameter of steel reinforcement}$; $s = \text{clear spacing of transverse steel reinforcement}$; $f_{op}$ is defined as $f_{op} = f_v$ or:

$$f_{op} = \frac{E_s}{\rho_v m^2(510 - 390m) + 40m + 670}, \text{ if } \rho_v \leq \left(\frac{4m + 67 - 0.1E_v/f_{cr}}{39m - 51}\right) \frac{1}{m^2} \quad (2.38.1)$$

where $m = E_{op}/E_v$; $E_s$ and $E_v$ are the elasticity moduli of steel confinement and unconfined concrete, respectively. $\varepsilon_{cc}$ is calculated:
where $\varepsilon_{co}$ = concrete compressive strain corresponding to unconfined concrete; $\varepsilon_y$ = yield strain of the transverse steel reinforcement.

It should be appointed that the confined concrete strain was derived based on an elastic-perfectly plastic steel behaviour. The shape of the steel stress-strain curve has no influence on the confined concrete strength. However, due to the path dependency of the concrete deformations, the shape of the steel stress-strain curve affects the confined concrete strain at peak stress. The lateral steel stress at concrete peak stress is not assumed to be equal to the yield stress. For the case of rounded stress-strain curve, the yield strain of steel is taken at the point of maximum stress. In general, the confinement’s enhancement of the confined concrete strength and the confined concrete peak strain is more pronounced for NSC than for HSC. The confined peak strain is more appropriate for unconfined concrete with strength ranging from 25 MPa to 73 MPa and for elastic-perfectly plastic lateral steel with yield strength not greater than 900 MPa (Eid et al. 2010).

The post peak branch ($\varepsilon_c > \varepsilon_{co}$) is a modified expression of the fractional relationship originally proposed by Sargin (1971) and adopted by Ahmad and Shah (1982):

$$\frac{\varepsilon_c}{\varepsilon_{co}} = 10.8 \rho_y \varepsilon_y^{0.5} - 0.021 + \sqrt{1.043 + \left(82m - 165\right) \rho_y^{0.85} \varepsilon_y^{0.5}}$$  \hspace{1cm} (2.38.2)

The post peak branch ($\varepsilon_c > \varepsilon_{co}$) is a modified expression of the fractional relationship originally proposed by Sargin (1971) and adopted by Ahmad and Shah (1982):

$$f_c = \frac{a_1 \varepsilon_c + a_2 \varepsilon_c^2}{1 + a_1 \varepsilon_c + a_2 \varepsilon_c^2}$$  \hspace{1cm} (2.39)

$$a_1 = K \left[ 0.85 \varepsilon_{cu} l_{l_s} + 0.85 \frac{\varepsilon_{x,s}}{\varepsilon_{ce}} l_3 - 0.15 \varepsilon_{x,s} l_s \right]$$  \hspace{1cm} (2.39.1)

$$a_2 = K \left[ 0.15 \frac{\varepsilon_{cu}}{\varepsilon_{ce}} \left(1 + \frac{\varepsilon_{ce}^2}{\varepsilon_{cm}^2}\right) + 1.7l_1 - 0.85 \frac{\varepsilon_{x,s}}{\varepsilon_{ce}} l_3 \left(1 + \frac{\varepsilon_{ce}^2}{\varepsilon_{x,s}^2}\right) \right]$$  \hspace{1cm} (2.39.2)
\[ a_3 = \frac{K}{f_{cc}} \left[ \varepsilon_{tu1} \left( 0.85 \frac{\varepsilon_{cr}}{\varepsilon_{cr,0.85}} - 2 \right) + 0.15 \varepsilon_{cr,0.85} \left( 2 \frac{f_{cc}}{f_{cr}} - \frac{\varepsilon_{cr}}{\varepsilon_{tu}} \right) + 0.85 \frac{\varepsilon_{cr,0.85} \varepsilon_{tu} l_1}{\varepsilon_{cr}} \right] \tag{2.39.3} \]

\[ a_4 = \frac{K}{f_{cc}} \left[ 1.7 l_1 - l_1 \left( 0.85 \frac{\varepsilon_{cr}}{\varepsilon_{cr,0.85}} + \frac{\varepsilon_{cr,0.85}}{\varepsilon_{cr}} \right) + 0.15 \frac{\varepsilon_{cr}}{\varepsilon_{tu}} \left( 1 + \frac{f_{cc}}{f_{cr}} \varepsilon_{cr}^2 \right) \right] \tag{2.39.4} \]

where \( l_1 = f_{cr} / f_{cc} - 1 \); \( l_2 = \varepsilon_{cr} / \varepsilon_{cr,0.85} - 2 \); \( l_3 = f_{cc} / f_{cr} - 1.176 \); \( l_4 = \varepsilon_{cr} / \varepsilon_{tu} - 2 \)

\[ K = \frac{f_{cc}}{0.15 \varepsilon_{cr,0.85} \left( \frac{f_{cc}}{f_{cr}} - 1 \right) \left( 1 - \frac{\varepsilon_{cr,0.85}}{\varepsilon_{cr}} \right)} \tag{2.39.5} \]

\[ \frac{f_{cc}}{f_{co}} = 0.47 \rho_{vfy} - 0.021 - 0.521 S + \sqrt{4.62 \times 10^{-4} - 1.47 \rho_{vfy} S + 4.97 \rho_{vfy} + 0.224 S + 0.272 S^2} \tag{2.39.6} \]

where \( S = 1/(0.021 f_{co} + 2.39) \); \( \rho_{vfy} = \rho_{fy} / f_{co} \)

\[ \varepsilon_{cr} = \varepsilon_{lim} - \frac{\nu}{B_z} \sqrt{\left( \frac{\nu}{B_2} - \varepsilon_{lim}^2 \right)^2 - \frac{B_1}{B_2}} \tag{2.39.7} \]

\[ \varepsilon_{lim} = \frac{1 - \nu}{\nu E_e} \sigma_{ut} - \varepsilon_{cr} \tag{2.39.8} \]

where \( \varepsilon_{cr} \) = cracking strain of concrete in direct tension; \( \nu \) = Poisson’s ratio; \( \sigma_{ut} = 0.47 \rho_{fy} \); \( B_z = (1 - 2 \nu) \varepsilon_{cr} / \left( \varepsilon_{cr} - \varepsilon_{lim}^2 \right)^2 \)

\[ B_3 = 2 \varepsilon_{sf} - \frac{0.94 (1 - 2 \nu) \varepsilon_{cr}}{E_e} \rho_{vfy} f_{co} + B_2 \left( \varepsilon_{lim}^2 \right) \tag{2.39.9} \]

where \( \varepsilon_{sf} \) = fracture strain of the lateral steel (negative sign denotes tension strain).

In terms of the peak stress of confined concrete, this model shows good agreement with the tested specimens by Li et al. (2001), Mander et al. (1988a), Razvi and Saatcioglu (1999). It is recommended that this model can be extended to predict the behaviour columns under constant axial load and reverse cyclic lateral loading (Eid et al. 2010).
2.5 Models for Wire Mesh Confined Concrete

Wire mesh has been used in column jacketing for strengthening existing columns. In relation to loading, there are two ways for applying column jacking: if jacket has the same length as the original column, the jacket is loaded directly; the jacket is loaded indirectly if there is gap between the ends of the original member and the jacket. It should be noted that in the second case indirectly loaded jacket can carry some load through friction.

Numerous studies have been carried on using wire mesh for column jacketing or internal reinforcement. In the following section, different models for predicting the ultimate load of wire mesh jacketed or reinforced concrete are presented. Both circular and rectangular section shapes are included. However, the former is the focus in this study.

2.5.1 Wire Mesh Confined Rectangular Columns

In the past wire mesh was commonly applied to low strength mortar or normal strength concrete (NSC) for strengthening or rehabilitating existing structural members. It was recommended that the wire-mesh-confined concrete had a considerable strength increase due to confinement (Singh et al. 1988).

Mansur and Paramasivam (1990) investigated ferrocement box sections with and without mortar infill loaded concentrically and eccentrically. In total 54 short 100 mm × 100 mm square (100 mm × 100 mm) columns with a height of 500 mm or 600 mm were tested. Mortar had a compressive strength of 40 MPa, approximately. The mortar shell was 20 mm thick and the protective cover was between 4-5 mm. Two methods were proposed to estimate the ultimate load. The first comprises two components: the contribution of mortar and steel, shown as Eq. 2.40; the second considers the contribution of wires negligible, shown as Eq. 2.41.

\[
P_u = 0.67 f'_m \left( A_g - A_s \right) + A_s f_{ys} \tag{2.40}
\]
\[ P_u = f_f A_g \]  

where \( f_u \) = cube crushing strength of mortar; \( A_g \) = gross cross-sectional area of the column; \( A_s \) = areas of steel in the direction of loading; \( f_{ys} \) = yield strength of reinforcing steel, \( f_f \) = ratio of the compressive strength 20 mm thick ferrocement plates to that of 100 mm cubes.

Ganesan and Anil (1993) investigated steel reinforced square columns (150 mm × 150 mm × 750 mm) confined with wire mesh shell. The ferrocement shell (20 mm thick) was cast first and used as formwork for casting concrete columns. Eq. 2.42 was proposed for wire mesh and steel reinforced concrete columns with rectangular cross section.

\[ \frac{P}{\sigma_{cu} bd} = 0.0654(V_f \rho_v) + 0.6366 \]  

where \( b, d \) = lateral dimensions of column, \( P \) = the ultimate load, \( \sigma_{cu} \) = cube strength of concrete, \( V_f \) = volume fraction of mesh reinforcement, \( \rho_v \) = volumetric ratio of transverse reinforcement. It is noted that the compressive strength of mortar and the reinforcement strength are not taken into account in this equation.

Fahmy et al. (1999) rehabilitated square columns using ferrocement. In total 24 reinforced NSC 100 mm × 100 mm × 1000 mm columns were first loaded failure up to 67% or 85% of the ultimate load and then repaired with 10 mm thick ferrocement jacket. It was confirmed that under the short term loading conditions, all the repaired columns gained more than the corresponding original ultimate strengths. The level of damage occurred before retrofitting affects the ultimate load of the repaired column. Eq. 2.43 was proposed to predict the ultimate load of ferrocement strengthened concrete columns.

\[ (\frac{P_u}{\sigma_{cu}})_{\text{repaired}} = A_c f_{cu} + A_t f_{ts} + A_{st} f_{st} + A_{sw} f_{sw} \]  

(2.43)
where \( A_c \) = concrete cross-sectional area; \( A_{st} \) = cross-section area of longitudinal bars, \( f_{ys} \) = yield stress of longitudinal bars, \( A_{fer} \) = cross-section area of the ferrocement jacket, \( f'_m \) = compressive strength of mortar, \( A_{ferw} \) = sum of cross-section areas of reinforcing wires, \( f_{yw} \) = yield strength of mesh.

### 2.5.2 Wire Mesh Confined Concrete with Circular Section

Based on the test results of 144 standard NSC cylinders, Waliuddin and Rafeeqi (1994) proposed Eq. 2.44 to estimate the compressive strength of mesh confined concrete. It is noted that the equation was based on woven square mesh with an aperture size of 6 mm.

\[
f_{cc} = f_{co} + Kf_{yw} \tag{2.44}
\]

where \( f_{cc} \) = theoretical compressive strength of concrete; \( f_{co} \) and \( f_{yw} \) are defined previously; \( K = K_mK_gK_p \), \( K_m \) = coefficient to account for the method of confinement, equal to 0.88 for the wrapping method; \( K_g \) = coefficient to account for the grade of concrete, taken as 1 in their study; \( K_p \) = coefficient to account for the number of wire mesh layers, equal to 35\( pK_r \); \( p \) = volume fraction of transverse wires taken over shell area and equals to \( V_{st} \); \( K_r \) = ratio of cross-sectional and surface area of shell. The final equation is presented as Eq. 2.45. It is noted that the mortar compressive strength and cover thickness were not indicated.

\[
f_{cc} = f_{co} + 35pK_mK_rf_{yw} \tag{2.45}
\]

Kondraivendhan and Pradhan (2009) investigated 42 cylindrical specimens, with a diameter of 150 mm and a height of 900 mm with a range of compressive strength. The 15 mm thick ferrocement shell contained one layer of chicken wire mesh. The researchers recommend that Waliuddin and Rafeeqi (1994)’s approach for estimating the confined compressive strength.

Kaushik and Singh (1997) investigated 60 \( \Omega 150 \times 300 \) mm cylindrical specimens cast with 1-3 layers of wire mesh and various amounts of longitudinal reinforcement. The
confined concrete strength, the corresponding strain and the ultimate load are presented in Eq. 2.46 - Eq. 2.49, respectively.

\[
\frac{f'_{cc}}{f_{co}} = 1 + k_1 \frac{f_{yw}}{f_{cc}} \tag{2.46}
\]

\[
\varepsilon_{cc} = \varepsilon_{co} \left(1 + k_1 \frac{f_{yw}}{f_{cc}}\right) \tag{2.47}
\]

\[
P_M = \pi \left( R^2 f_{co} + R_4 k_i V_f (R - R_i) \times f_{yw} / 2 \right) + A_{st} \left( f_{yw} - f_{cc} \right) \tag{2.48}
\]

\[
V_f = 2\pi d_w^2 \times nm / \left( s_w (R - R_c) \right) \tag{2.49}
\]

where \( f_{co} \) and \( f'_{cc} \) = compressive strength of confined and unconfined concrete respectively; \( \varepsilon_{co} \) and \( \varepsilon_{cc} \) = strain corresponding to \( f_{co} \) and \( f'_{cc} \), respectively; \( f_{yw} \) = lateral confining pressure contributed by wire mesh; \( k_1 = \) strength increase factor, taking as 4; \( k_2 = 20; R, R_c \) and \( R_M = \) radii of the column, the core concrete and the mesh layers (mean radii), respectively; \( A_{st} \) defined as earlier; \( f_{yw} = \) yield stress of single wire strand, \( V_f = \) volume fraction of wire mesh determined by Eq. 2.49, \( d_w = \) radius of wires, \( n = \) number of mesh layers; \( s_w = \) wire spacing. It is noted that the diameter of the confined concrete core and the cover thickness were not clear.

Ho et al. (2013) used different types of mortar with either one layer or three layers of wire mesh to replace the original concrete cover (25 mm thick). Three types of matrices were adopted: cementitious mortar, polymer-based matrix and epoxy-based matrix. The epoxy-based matrix exhibited high tensile strength. The reinforcement characteristics significantly influence the confinement effect and the failure mode of confined concrete. The load and elongation curves of this type of ferrocement were not presented.

In total 19 full-scale circular plain or reinforced concrete columns were tested under monotonic compression. It was found that plain concrete columns confined with the
improved mortar composite achieved 30-59% higher strength compared to the controls. The reinforced concrete columns with low volume ratio $\rho_s$ reached the peak strength comparable with columns having a significant higher volume ratio. Ho et al. (2013) proposed Eq. 2.50 and Eq. 2.51 to estimate the peak strength of RC columns confined with ferrocement. This method may particularly match the case of ferrocement with high tensile strength matrix.

$$ P = K(P^R + P^F) + A_f f_y $$

(2.50)

$$ K = 1 - 0.513 \left( \frac{f_y'}{f_p'} \right)^{0.150} $$

(2.51)

where $P^R$ = contribution by RC columns; $P^F$ = contribution by ferrocement; $K$ = factor for considering the interactions between steel hoop and ferrocement; $f_y'$ = 0.5$\rho_s f_{ys}$; $f_p' = 2T/d_{ext}$; $T$ = tensile strength of ferrocement plate from experiment; $d_{ext}$ = diameter of the concrete core measured at the centre line of mortar shell. The remaining notation is the same as mentioned earlier. It is noted that the compressive strength of mortar was not clear. $P^R$ and $P^F$, can be determined using the following equations:

$$ P^R = f'_{co} A_c \left[ 1 - 0.375 \left( \frac{s}{d_c} \right)^{0.480} + 847.385 \left( \frac{f_y'}{f'_{co}} \right)^{5.377} + 1.349 \left( \frac{f_y'}{f'_{co}} \right)^{0.405} \right] $$

(2.52)

$$ P^F = f'_{co} K_1 A_c $$

(2.53)

$$ K_1 = 0.064 \left( \frac{f_y'}{f'_{co}} \right)^{0.551} + 26.920 \left( \frac{f_y'}{f'_{co}} \right) $$

(2.54)

where $f'_{co}$ = concrete compressive strength of unconfined column. It is noted that the compressive strength of cover materials was not clear and a full axial load and displacement curve was not presented.

Kaish et al. (2015) investigated different sizes of cylindrical specimens strengthened with one and two layers of wire mesh. It was suggested that size difference can affect
the confinement effectiveness and wire mesh mortar composite can enhance the strength and ductility. Kaish et al. (2015) proposed the following equations for predicting the strength of wire mesh confined concrete.

\[ \frac{f_{cc}}{f_{co}} = 1 + 11.2 \frac{f_{lw}}{f_{co}} \]  

(2.55)

\[ f_{in} = \frac{2nA_wf_{yw}(H_j + s_w)}{(D_cH_s)_w} \]  

(2.56)

where \( f_{co}, f_{cc}, f_{lw}, n, s_w \) and \( D_c \) are as defined previously; \( A_w \) = cross-sectional area of single wire; \( H_j \) = height of ferrocement jacket.

It is noted that for large specimens it is reasonably to simplify the component \((H_j + s_w)\) with \( H_j \). To avoid mortar composite carrying direct axial load, the ferrocement jacket was finished 10 mm below or above the top and bottom ends of concrete core. In this way direct axial load can be avoided. However, mortar shell can carry some load indirectly through friction, especially for the case of roughened circumference.

2.5.3 Steel, Wire Mesh and Mortar Confined Concrete

Shang et al. (2005) investigated the behaviour of square steel reinforced concrete columns strengthened with steel mesh (diameter = 6.45 mm) and mortar composite under eccentrically monotonic loading. The results show that the capacity of ductility, deformation and energy dissipation of the strengthened concrete column specimens improved.

Xiong et al. (2011) conducted study on steel (including vertical steel bars), wire mesh and normal strength mortar confined NSC columns. The cylindrical concrete core was 450 mm high with a diameter of 105 mm. The mortar shell was 25 mm thick. Xiong et al. (2011) proposed Eq. 2.57-Eq. 2.58 for predicting the ultimate load carrying capacity of wire mesh and steel mat confined concrete columns.
$$P' = f'_{co} A_c = f'_{co} (1+1.98\lambda_t) A_c$$  \hspace{1cm} (2.57)

$$\lambda_t = \left( \mu_t f_{ys} + \mu_w f_{yw} \right) / f_{co}$$  \hspace{1cm} (2.58)

where \( f'_{co} \) = concrete compressive strength of unconfined column; \( A_c \) = area of concrete core; \( \lambda_t \) = confinement ratio; \( \mu_t \) = reinforcement ratio provided by transverse and longitudinal steel bars; \( f_{ys} \) = yield stress of steel confinement; \( \mu_w \) = reinforcement ratio provided by the transverse and longitudinal wires of mesh; \( f_{yw} \) = yield stress of wire.

As shown in the literature review, usually the compressive strength of mortar and protective cover thickness were not clear. This is because wire mesh was commonly used with low strength mortar and the contribution of matrix is considered negligible. A full axial load and axial displacement curve was not presented in most of the existing literature. This raises the question of when the reinforcement becomes effective. There is a lack of information on applying wire mesh with high strength cementitious matrix and the behavior of concrete strengthened by such a composite. Mortar can be economically modified to improve its properties. High strength cementitious matrices improve not only the load carrying capacity of the strengthened columns but also the interface strength. However, under axial compression with the utilisation of high strength concrete/mortar jacket, a more brittle behaviour is expected (Julio et al. 2005; Campione et al. 2014). To enhance ductile behaviour, a relatively thin jacket and cover is preferable. To employ the advantages of different materials, a jacketing consisting of steel hoops, wire mesh and modified high strength mortar seems to be a promising technology in terms of the load carrying capacity and ductile post-peak behaviour for concrete structure repair and rehabilitation.
2.6 Summary

This chapter provides a comprehensive review of existing literature about the latest application of wire mesh and mortar composite. The trend of applying modified high strength mortar and combining with steel transverse reinforcement to the conventional wire mesh and mortar composite was addressed. Models about each type of confinement were presented chronologically. A gap between this new composite and the limited understanding on its structural behaviour has been identified. In the following chapter, an experimental study on the axial compressive behaviour of wire mesh confined high strength concrete and median strength concrete will be presented.
CHAPTER 3  EXPERIMENTAL STUDY ON MESH STRENGTHENED HIGH STRENGTH CONCRETE AND MEDIUM STRENGTH CONCRETE

3.1 Introduction

Wire mesh and mortar composite has been adopted to enhance the performance of concrete members with low compressive strength. The purposes of the series of experiments, Experiment I and Experiment II, are to (1) explores the potential of extending the application of wire mesh to high strength concrete (HSC) and medium strength concrete(MSC); (2) develop an efficient method of forming wire mesh reinforcement. Wire mesh was installed in two different ways. For the first experiment, wire mesh was wrapped to the HSC core after concrete casting; while in the second experiment wire mesh cages were placed into the steel moulds before MSC cast. The investigation was focused on the axial load and axial deformation. The behaviour of strengthened specimens was investigated in terms of failure mode, the load carrying capacity, the ductility and energy absorption. Different approaches were adopted to estimate the ultimate load. The proposed equations provide estimation for the ultimate load with reasonable accuracy. It is noted that Experiment I and Experiment II were mainly carried out in the structural laboratory at Hong Kong Polytechnic University, unless otherwise specified.

3.2 Experiment I Concrete Casting

3.2.1 Trial Mix

Concrete used for this study was mixed in the laboratory. To investigate the mix design of high strength concrete, a trial cast was conducted first. The target compressive strength was 85 MPa. Standard Ø100 × 200 mm cylinders were cast with steel moulds for determining the concrete compressive strength. The tests of concrete compressive strength for the trial cast were conducted using Servo Tronic (Matest) at a speed of 0.6 MPa/s. This machine has a capacity of 3000 kN. The compressive strength of Ø100 × 200 mm cylinders was 38 MPa at 3 days, reached 45% of the target strength.
3.2.2 HSC Casting and Compressive Strength at 28 Days

After concrete trial mix, 12 Ø150 × 300 mm standard cylinders and three Ø100 × 200 mm cylinders were cast first with target strength of 85 MPa. The cast was in accordance with AS1012.8.1 (2000). The result of slump test was 173 mm, complying with AS1012.3.1 (1998).

Approximately four hours after casting, all the specimens were covered with a plastic sheet on top to prevent moisture loss and allow continual hydration of cement. After being removed from steel moulds, all the specimens were covered with plastic sheets and watered regularly until 28 days. It was observed some voids appeared on the specimens’ circumference, which indicated that vibration might not be sufficient during casting.

The tests of the concrete compressive strength tests were conducted using ServoTronic (Matest) at a speed of 0.6 MPa/s (same as the trial test). At 28 days the average compressive strength for the Ø100 × 200 mm cylinders and Ø150 × 300 mm cylinders was 74 MPa and 77 MPa, respectively.

3.3 Mortar Trial Mixes

The key challenge was to ensure mortar working together with the concrete core. Based on the existing literature, a number of mortar designs had been investigated. However, the amount of water reducer, which significantly affects the workability and strength of mortar, was not indicated in the literature. Mortar mix trials were focused on using basic materials available in the laboratory to obtain high strength with appropriate workability, which allowed mortar to be pressed through wire mesh and stay on the circumference. A number of trial mixes were conducted using 70 mm cubic steel moulds. Portland cement ASTM Type I and natural sand were used for mortar mix.

3.3.1 Mortar Compressive Strength at 7 Days

The mix proportion and the results of four trials are presented in Table 3.1. The mix ratio was with respect to cement, sand, water and superplasticiser. The
superplasticiser used in this experiment was ADVA1.9. A standard manual flow table was used to measure the consistency of mortar, in compliance with ASTM C230/C230M-2008 and ASTM C1437-2007, respectively. Three 70 mm cubic specimens of each batch were tested at Day7. As shown in Table 3.1, water cement ratio can significantly affect the strength and workability. With the increase of w/c ratio, the strength decreased accordingly. In addition, the results indicate that superplasticiser has a significant influence on mortar strength and workability.

| Mortar Mix | Mix ratio (by weight of cement) | Results | | | |
|---|---|---|---|---|
| | Cement | Sand | Water | Superplasticiser* (%) | Flow (%) | Compressive Strength at 7 days (MPa) |
| MTB1 | 1 | 2.71 | 0.41 | 6.6 | 117 | 48.03 |
| MTB2 | 1 | 2.65 | 0.41 | 1.7 | 89 | 62.55 |
| MTB3 | 1 | 2.66 | 0.36 | 1.8 | 23 | 67.23 |
| MTB4 | 1 | 2.65 | 0.51 | 0.6 | 89 | 49.17 |

Note: * ADVA1.9

3.3.2 Flow Table Test

Through the trial tests, the mix design ratio of 1:2.5:0.35 was adopted for the mesh-confined specimens. Ideally, the diameter of the mortar mass should be within the range from 135 mm to 150 mm. Large variation from the considered range will affect the bond between mortar and concrete. Some unfavorable examples are shown in Fig. 3.1

Fig. 3.1(a) shows that mortar was too liquid to stay on the concrete core with no presence of formwork, while mortar in Fig. 3.1 (b) did not seem workable enough for applying onto the wire mesh confined concrete core.
3.4 Different Types of Wire Mesh

A number of galvanized wire meshes with different openings were investigated, including 10 mm × 10 mm wire mesh (E10WM), 18 mm × 18 mm wire mesh (E18WM), 25 mm × 25 mm wire mesh (E25WM), 12.7 mm × 12.7 mm wire mesh (E12.7WM) and 14 mm × 14 mm wire mesh (E14WM). Due to the constraints of test equipment, only single wire samples were tested. All the tensile strength tests were conducted using Testometric, M500_50 kN, at a speed of 0.3 mm/min. The test results of the five types of wire mesh are presented in Table 3.2.

<table>
<thead>
<tr>
<th>Mesh Type</th>
<th>Opening Size (mm)</th>
<th>Average Thickness (mm)</th>
<th>Average Ultimate Load (N)</th>
<th>Average Ultimate Stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E10WM</td>
<td>10 × 10</td>
<td>0.63</td>
<td>192</td>
<td>626</td>
</tr>
<tr>
<td>E12.7WM</td>
<td>12.7 × 12.7</td>
<td>0.55</td>
<td>222</td>
<td>947</td>
</tr>
<tr>
<td>E14WM</td>
<td>14 × 14</td>
<td>0.92</td>
<td>330</td>
<td>499</td>
</tr>
<tr>
<td>E18WM</td>
<td>18 × 18</td>
<td>0.78</td>
<td>301</td>
<td>640</td>
</tr>
<tr>
<td>E25WM</td>
<td>25 × 25</td>
<td>0.89</td>
<td>530</td>
<td>851</td>
</tr>
</tbody>
</table>

Table 3.2 shows that E12.7WM had the highest value in ultimate stress, outweighing its counterparts E14WM and E10WM. Therefore, E12.7WM was chosen as mesh reinforcement for this study.

3.5 Specimen Preparation

The parameters investigated in this study are the number of wire mesh layers and the nominal thickness of mortar cover. Eight Ø150 × 300 mm HSC specimens were reinforced with one or two layers of wire mesh and mortar cover. The protective cover was not identical. The nominal thickness of mortar cover was 5 mm and 10 mm, respectively. All the specimens were divided into four categories and each category contained two replicates. The cross sections of the confined specimens for each category are shown in Fig. 3.2 and Table 3.3. Control 1 and Control 2 served as reference; neither wire mesh nor mortar was applied. Wire mesh confined specimens were indicated with alphanumeric labels, in which L and C stand for layer and cover,
respectively. The Arabic number in front of the capital letter indicates the number of mesh layers and the nominal thickness of cover, respectively. The last Arabic number shows the number of replicates. For instance, the specimens confined with one layer of wire mesh and having a nominal 5 mm thick mortar cover were marked as 1L5C1 and 1L5C2.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Diameter*</th>
<th>Diameter†</th>
<th>Diameter</th>
<th>Height</th>
<th>Cover</th>
<th>No. of Mesh Layers</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$D_c$ (mm)</td>
<td>$D_m$ (mm)</td>
<td>$D$ (mm)</td>
<td>$H$ (mm)</td>
<td>$t$ (mm)</td>
<td></td>
</tr>
<tr>
<td>Control1</td>
<td>150</td>
<td>150</td>
<td>160</td>
<td>300</td>
<td>5</td>
<td>1</td>
</tr>
<tr>
<td>Control2</td>
<td>150</td>
<td>150</td>
<td>160</td>
<td>300</td>
<td>5</td>
<td>1</td>
</tr>
<tr>
<td>1L5C1</td>
<td>150</td>
<td>150</td>
<td>160</td>
<td>300</td>
<td>5</td>
<td>1</td>
</tr>
<tr>
<td>1L5C2</td>
<td>150</td>
<td>150</td>
<td>160</td>
<td>300</td>
<td>5</td>
<td>1</td>
</tr>
<tr>
<td>1L10C1</td>
<td>150</td>
<td>150</td>
<td>170</td>
<td>300</td>
<td>10</td>
<td>1</td>
</tr>
<tr>
<td>1L10C2</td>
<td>150</td>
<td>150</td>
<td>170</td>
<td>300</td>
<td>10</td>
<td>1</td>
</tr>
<tr>
<td>2L5C1</td>
<td>150</td>
<td>153</td>
<td>170</td>
<td>300</td>
<td>5</td>
<td>2</td>
</tr>
<tr>
<td>2L5C2</td>
<td>150</td>
<td>153</td>
<td>170</td>
<td>300</td>
<td>5</td>
<td>2</td>
</tr>
</tbody>
</table>

Note:
* Diameter of the concrete core; † The overall dimension of the mean of the wire mesh layers

As shown in Table 3.3, the letter combination $D_c$ is the diameter of HSC core; the letter combination $D_m$ is the overall dimension of the mean of the wire mesh layers; the letter $D$ stands for the diameter of strengthened specimen. The diameter for the specimens confined with one layer and two layers of wire mesh and 5 mm thick mortar cover was 160 mm and 170 mm, respectively. The diameter for the specimens confined with one layer of wire mesh and 10 mm thick mortar cover was 170 mm. For the convenience, $D_m$ was equal to the diameter of the concrete core if one layer of wire mesh was applied. If two layers of wire mesh were applied, $D_m$ is the overall dimension of the mean of the wire mesh layers, which was approximately 153 mm. Cover in Table 3.4, means the nominal cover. In the case of one layer of wire mesh the actual protective cover almost equals to the normal thickness, while in the case of two layers of wire mesh, the actual protective cover was slightly greater than the nominal value. For instance, the actual protective cover of Specimens 2L5C1 and 2L5C2 was approximately 7 mm as the overall dimension measured between the outside of the outermost lateral wires was 156 mm. All the specimens had an identical height of 300 mm and were tested under concentric loading.
After cured for 28 days, the concrete specimens were roughened evenly with an electrical chisel. Wire mesh was manually wrapped onto the concrete core layer by layer. For each layer, the overlap was approximately 76 mm (6 grids, $6 \times 12.7$ mm) as shown in Fig. 3.3(b). For the specimens confined with two layers of wire mesh, the overlaps of each layer were opposite to each other. To ensure that the overlap would not loosen or slide from each other, it was tied with steel wire (having a similar diameter as the wire mesh) at a spacing of 63 mm along the length of overlap on each side. At the end extra steel wires were applied to tie up wire mesh. A pair of plastic tags was used to mark the overlap area for further the investigation after test. Hose clips were used to facilitate fastening the wire mesh layers. After installing the wire mesh and removing the clips, the confined specimens were placed vertically on a levelled and hard timber base. Mortar was firmly plastered onto the circumference.
of the concrete core. Plywood plates (20 mm thick) with a smooth circular opening corresponding to the diameters of the strengthened specimens were used to control the geometry. Fig. 3.3 shows the main procedures of strengthening HSC specimens.

Mortar was cast in three batches due to the equipment constrain. The materials used for the mortar matrixes were the same as mentioned in the previous section. For each batch, three 70 mm cubic mortar specimens were prepared for the compressive strength test. It is noted that the cubic specimens had a normalised dimension of 70 mm × 70 mm ×70 mm. The tests of mortar compressive strength were conducted using ServoTronic (Matest) at a speed of 0.6 MPa/s. The results of the mortar compressive strength tests for all the confined specimens are presented in Table 3.4.

<table>
<thead>
<tr>
<th>Mortar Mix</th>
<th>Mix ratio (by weight of cement)</th>
<th>Curing (Days)</th>
<th>Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cement</td>
<td>Sand</td>
<td>Water</td>
</tr>
<tr>
<td>FMB1</td>
<td>1</td>
<td>2.58</td>
<td>0.37</td>
</tr>
<tr>
<td>FMB2</td>
<td>1</td>
<td>2.55</td>
<td>0.33</td>
</tr>
<tr>
<td>FMB3</td>
<td>1</td>
<td>2.60</td>
<td>0.27</td>
</tr>
</tbody>
</table>

Note: * ADVA1.9.
To strengthen each end of the wire mesh confined specimens, two layers of 40 mm wide CFRP strips were glued to the ends using Epoxy Sika 300 provided by a local company, which is commonly used for CFRP jacketing. Part A and Part B of the Sika 300 were continuously mixing for 5 minutes, and then brushed on both sides of CFRP strips, which were applied to the ends in a continuous manner. After the CFRP strips were applied, 48 hours were allowed for drying. To ensure even ends, each end of the specimens was capped with high strength plaster before testing.

Two strain gauges with a gauge length of 100 mm were placed opposite to each other in a vertical direction at the middle height of each specimen. To ensure a good bond condition between the strain gauge and the concrete underneath, all the areas for strain gauges were smoothed with sand paper and then coated with adhesive concrete pre-coating glue (manufactured by Tokyo Sokki Kenkyujo Co. Ltd.). For the wire mesh strengthened specimens, a method of measuring the post cracking strain was also investigated in this study. Due to the nature of wire mesh reinforcement, relative small and large numbers of cracks were expected, which could damage strain gauges at the early stage. To limit the damage, a geotechnical membrane strip with 20 mm width was glued onto the circumference after the two 100 mm strain gauges were applied. Four 50 mm strain gauges equally spaced were glued onto the membrane. However, this method is not recommended based on the investigation.

3.6 Specimen Testing and Failure Modes

All the specimens were tested under concentric loading using MTS 643, which has a capacity of 4600 kN. The testing was conducted by displacement control, carried out at a speed of 0.15 mm/min. Two Linear Variable Differential Transducers (LVDTs) were firmly placed to a steel frame, which was attached to the test specimen for recording the axial displacement. The gauge length of each LVDT was 120 mm. The tested specimens are shown in Fig. 3.4.
All the specimens were tested to failure. Three types of failure mode were observed: 1) scale like peeling, presented by Specimen 1L5C2; 2) large vertical and declined cracks, presented by Specimen 1L10C2 and 3) typical bulge at the mid height, presented by Specimen 1L10C1. In general, lateral wires failed in tension while vertical wires were buckled.

Concrete controls failed quickly when vertical cracks developed throughout the specimens. For wire mesh-confined specimens, the deformation was extensive. Fine vertical cracks initially appeared. Cracks continued extending and increasing in width. Large vertical cracks developed throughout the height of the specimens while lateral cracks also appeared. Vertical cracks were the dominated phenomenon. In some cases vertical cracks developed in a symmetrical fashion at the gauge length round the surface. However, scale pattern also appeared. During the test, with the increase of crack quantity and width in both directions, pieces of mortar cover spalled off from the surface. Lateral wires ruptured with a snapping sound. It is noted
that failure of wire mesh confined specimens was not associated with huge sound as occurred in high strength concrete controls.

### 3.7 Test Results and Discussion

#### 3.7.1 The Axial Load and Axial Displacement Curves

The axial load and axial displacement curves are illustrated in Fig. 3.5 and the results of the tested specimens are summarised in Table 3.5. In Table 3.5 the volume fraction of wire mesh in the lateral direction is indicated as $V_{\rho}$, with respect to the mortar shell (Naaman, 2000); the volumetric ratio of lateral wires relative to the core is noted as $\rho_w$. It is noted that the axial load and axial displacement curves were generated based on the reading of the plate. Due to cracking, for some specimens the LVDT readings became not consistent after the ultimate load.

![Fig. 3.5 Axial load and axial displacement](image-url)
Table 3.5 Test Results of Experiment I

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Mortar Strength $f_m$ (MPa)</th>
<th>Volume Fraction $V_f$ (%)</th>
<th>Volume Ratio $\rho_w$ (%)</th>
<th>Yield Load $P_y$ (kN)</th>
<th>Ultimate Load $P_u$ (kN)</th>
<th>Average Load $P_a$ (kN)</th>
<th>Ultimate Displ. $\delta$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control 1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1309</td>
<td>1375</td>
<td>1472</td>
<td>1.341</td>
</tr>
<tr>
<td>Control 2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1471</td>
<td>1570</td>
<td>1675</td>
<td>1.576</td>
</tr>
<tr>
<td>1L5C1</td>
<td>57.6</td>
<td>0.38</td>
<td>0.05</td>
<td>1565</td>
<td>1676</td>
<td>1659</td>
<td>1.527</td>
</tr>
<tr>
<td>1L5C2</td>
<td>57.6</td>
<td>0.38</td>
<td>0.05</td>
<td>1432</td>
<td>1641</td>
<td>1517</td>
<td>1.989</td>
</tr>
<tr>
<td>1L10C1</td>
<td>66.3</td>
<td>0.19</td>
<td>0.05</td>
<td>1573</td>
<td>1728</td>
<td>1623</td>
<td>1.719</td>
</tr>
<tr>
<td>1L10C2</td>
<td>66.3</td>
<td>0.19</td>
<td>0.05</td>
<td>1436</td>
<td>1517</td>
<td>1573</td>
<td>1.334</td>
</tr>
<tr>
<td>2L5C1</td>
<td>70.6</td>
<td>0.38</td>
<td>0.10</td>
<td>1609</td>
<td>1737</td>
<td>1725</td>
<td>1.638</td>
</tr>
<tr>
<td>2L5C2</td>
<td>70.6</td>
<td>0.38</td>
<td>0.10</td>
<td>1568</td>
<td>1713</td>
<td>1683</td>
<td>1.811</td>
</tr>
</tbody>
</table>

Note: *Displ.* = displacement; $V_f$ = volume fraction of wires in the lateral direction with respect to the mortar shell; $\rho_w$ = volumetric ratio of lateral wires with respects to the core.

It was observed that the sudden and brittle cover spalling, which is particularly associated with HSC (Bae and Bayrak 2003), did not occur. This is due to the uniform dispersion of wire mesh, which transforms the brittle mortar into a composite of ductility (Mansur 1987). As shown in Fig. 3.5, the load carrying capacity of Control 1 and Control 2 dropped immediately after passing the ultimate load, evidenced by a nearly vertical line. Compared to the reference specimens, the load carrying capacity was improved 10% -17% by using wire mesh and high strength mortar. For the specimens confined with wire mesh and high strength mortar, the load dropped immediately after passing the ultimate load. However, the load dropped less sharp than that of the controls and a significant change in the slope of the descending branch of curves approximately occurred at 60% -50% past the ultimate load. This indicates that the strength degradation rate reduced.

Compared to Specimens 1L5C1 and 1L5C2, the ultimate load of Specimens 2L5C1 and 2L5C2 increased less than 5% with the increase in the mortar strength, matrix area and the number of wire mesh layers. The increment in the load carrying capacity is likely related to the increase of the area and the compressive strength of mortar shell. In other words, compared to the controls, the significant increase in the ultimate load suggests that high strength mortar matrices have significant influence on the ultimate load, particularly for the case that the strengthening jacket contains only wire mesh as confinement. It is noted there an apparent variation in the ultimate
load between Specimen 1L10C1 and Specimen1L10C2. This might be caused by the imperfection of the HSC core of Specimen1L10C2. It was found there were voids in the concrete core, as shown in Fig. 3.6. It could also partially due to the varying nature of concrete (Guo and Shi 2003).

Fig. 3.6 Imperfection on concrete core confined with one layer of wire mesh

Fig. 3.5 also suggests that the energy absorption capacity of confined specimens increased during the post peak load stage. To quantify energy absorption, the areas under the axial load and axial displacement curves (AUC) were calculated (Ganesan and Anil 1993). As the testing for all the specimens did not finish at the same peak load, 50% of the post peak load was set as the boundary of AUC for each specimen. For the purpose of comparison, a factor known as energy absorption index was adopted. The energy absorption index is determined as: 1) computing AUC up to 50% of the post peak load for each specimen; 2) taking the average AUC of Control 1 and Control 2 as the reference; 3) dividing AUC of the confined specimens by the reference. The energy absorption index is designated as $I_{50}$.

The energy absorption indexes are shown in Table 3.6, which indicates that wire mesh has advantage in energy absorption. However, for Specimens 2L5C1(2), with the increase in the mesh layers, the energy absorption indexes ($I_{50}$) decreased slightly, compared to Specimens 1L5C1(2). It is noted that this kind of discrepancy also appeared in Wang et al. (2015)’s study.

A few factors could contribute to this discrepancy, such as the characteristics of constituent materials, weak confinement effect and scatter of test results. Firstly, HSC and high strength mortar were used in the first series of experiments; the wire
The mesh used had the smallest diameter, among the three types of wire mesh used in the present study. The volume ratios of transverse wires for Specimens 1L5C1(2) and 2L5C1(2) were 0.05% and 0.10%, respectively, which both provided only marginal confinement. Secondly, for HSC, the relative increase in lateral strains in the inelastic range is less, compared to NSC. The expansion of concrete core is accordingly lower. Combined with the very limited amount of wire mesh, the confinement effect was weak and mesh might only become effective during the late stage of loading. Thirdly, HSC has a relatively large scatter compared to NSC (ACI 363R-1992). Additionally, the increase in the mortar strength and thickness leads to a more brittle behaviour (Júlio et al. 2005; Campione et al. 2014), which may also influence the scatter of test results.

Moreover, it is also found that the strain readings from LVDT and the strain gauges in the vertical direction were not consistent for all the specimens. For example, for Specimens 1L5C2 and 2L5C1, the two types of readings were consisted up to the ultimate load and for Specimen 1L10C1, the two types of readings were very close up to 72.9% of the ultimate load; while for the other specimens the two readings became inconsistent at the early stage. In general the strain measured from LVDT increased faster than the readings from strain gauges.

Table 3.6 Area under the axial load and axial displacement curve

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Area under the axial load and deflection curve (AUC), kN.mm</th>
<th>Energy absorption Index $I_{50}$</th>
<th>Average $I_{50}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control 1</td>
<td>1034</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Control 2</td>
<td>1432</td>
<td>1*</td>
<td>-</td>
</tr>
<tr>
<td>1L5C1</td>
<td>2217</td>
<td>1.798</td>
<td></td>
</tr>
<tr>
<td>1L5C2</td>
<td>2112</td>
<td>1.713</td>
<td>1.76</td>
</tr>
<tr>
<td>1L10C1</td>
<td>2207</td>
<td>1.790</td>
<td></td>
</tr>
<tr>
<td>1L10C2</td>
<td>1523</td>
<td>1.236</td>
<td>1.51</td>
</tr>
<tr>
<td>2L5C1</td>
<td>1997</td>
<td>1.620</td>
<td></td>
</tr>
<tr>
<td>2L5C2</td>
<td>2035</td>
<td>1.651</td>
<td>1.64</td>
</tr>
</tbody>
</table>

Note: *Take the average value $I_{50}$ of Control1 and Control2 as the reference.
3.7.2 Ductility

As a perforable characteristic of any structural material, ductility provides warning of failure (Nawy 2003). Two methods were adopted for ductility analysis. The first method refers to the ratio of axial displacement at the ultimate load ($\Delta_u$) to the axial displacement at the yield load ($\Delta_y$), expressed as Eq. 3.1. The yield load is the load corresponding to the axial displacement at the limit of the elastic behaviour of specimens ($\Delta_y$). $\Delta_y$ is determined following Pessiki and Pieroni’s approach (1997): (1) a best-fit line to the linear portion of the axial load-axial displacement curve was implemented, which can be approximately obtained by connecting the origin coordinate and the coordinate at the 75% of the ultimate load (Foster and Attard 2001; Kazemi and Morshed 2005); (2) this line was then extended to intersect with the ultimate load; (3) the displacement corresponding to this intersection was marked $\Delta_y$, as shown in Fig 3.7.

$$\mu_y = \frac{\Delta_u}{\Delta_y} \quad (3.1)$$

The second method is based on Pessiki and Pieroni’s approach (1997), which refers to the ratio of the post-yield axial displacement at 85% of the ultimate load ($\Delta_{85}$) to the axial displacement at the yield point ($\Delta_y$). The expression of the second method is represented as Eq. 3.2 and illustrated in Fig. 3.7. The ductility results are shown in Table 3.7 and Fig. 3.8.

$$\mu_{85} = \frac{\Delta_{85}}{\Delta_y} \quad (3.2)$$
Fig. 3.7 Pessiki and Pieroni’s (1997) ductility approach

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield Load $P_y$ (kN)</th>
<th>Ultimate Load $P_u$ (kN)</th>
<th>Axial Displacement</th>
<th>Ductility Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\Delta y$ (mm)</td>
<td>$\Delta u$ (mm)</td>
<td>$\Delta_{.85}$ (mm)</td>
<td>$\Delta_u/\Delta_y$</td>
</tr>
<tr>
<td>Control 1</td>
<td>1309</td>
<td>1375</td>
<td>1.239</td>
<td>1.341</td>
</tr>
<tr>
<td>Control 2</td>
<td>1471</td>
<td>1570</td>
<td>1.382</td>
<td>1.576</td>
</tr>
<tr>
<td>1L5C1</td>
<td>1565</td>
<td>1676</td>
<td>1.345</td>
<td>1.527</td>
</tr>
<tr>
<td>1L5C2</td>
<td>1432</td>
<td>1641</td>
<td>1.562</td>
<td>1.989</td>
</tr>
<tr>
<td>1L10C1</td>
<td>1573</td>
<td>1728</td>
<td>1.411</td>
<td>1.719</td>
</tr>
<tr>
<td>1L10C2</td>
<td>1436</td>
<td>1517</td>
<td>1.218</td>
<td>1.334</td>
</tr>
<tr>
<td>2L5C1</td>
<td>1609</td>
<td>1737</td>
<td>1.385</td>
<td>1.638</td>
</tr>
<tr>
<td>2L5C2</td>
<td>1568</td>
<td>1713</td>
<td>1.485</td>
<td>1.811</td>
</tr>
</tbody>
</table>
As shown in Table 3.7 and Fig. 3.8, compared to the controls, the two ductility ratios, $\mu_s$ and $\mu_{85}$, improved modestly for wire mesh reinforced specimens. The average value of $\mu_s$ for the specimens confined with one layer of wire mesh with 5 mm cover, one layer of wire mesh with 10 mm cover and two layers of wire mesh with 5 mm cover was increased 8.4%, 4.1% and 8.0%, respectively. The average value of $\mu_{85}$ for the specimens confined with one layer of wire mesh with 5 mm cover, one layer of wire mesh with 10 mm cover and two layers of wire mesh with 5 mm cover was increased 6.4%, 4.8% and 7.1%, respectively.

3.7.3 Ultimate load estimation for wire confined HSC

Under concentric loading it is reasonable to consider wire mesh as steel reinforcement. Lateral wires accordingly could be considered as closely placed spiral or circular hoops with small diameter. As the diameter is small, the load contribution of wires in the vertical direction is negligible. Applying the concept of effective confined area as indicated by a number of researchers at the lateral wire level, the confinement is maximum and reduced most at the mid height between the lateral wires spacing (Park and Paulay 1975; Sheik and Uzumeri 1980; Maner et al. 1988a; Foster et al. 1998). As the aperture size of wire mesh is small, the spacing of the lateral wires was small. Accordingly, the effective confined area is relatively large and even, as shown in Fig. 3.9. However, due to the small cross section of single
wires, confinement is limited in terms of the magnitude. The confining mechanism is illustrated in Fig. 3.10. It is noted that the contribution of mortar tensile strength is considered negligible in this study (AS3600 2009).

Similar to steel confined concrete as indicated in Chapter 2, the effective confining pressure of wire mesh confined HSC can be expressed as following (AS3600-2009):

\[
 f_{r,\text{eff}} = \left(1 - \frac{s_w}{2D_m}\right)^2 \left(\frac{2nA_w f_{yw}}{D_m s_w}\right)
\]

where \((1 - s_w/2D_m)^2\) is a simplified effectiveness factor accounting for the arrangement of the lateral reinforcement, noted as \(k_{ew}\); \(2nA_w f_{yw} / D_m s_w\) is the average confining pressure on the core cross-section taken at the level of lateral reinforcement, noted as \(f_r\); \(s_w\) is the wire spacing; \(D_m\) is the overall dimension of the mean of the wire mesh layers; \(n\) is the number of wire mesh; \(A_w\) is the cross-sectional area of single steel
wire; \( f_{yw} \) is the yield stress of steel wire. Due to the equipment constrains, the load
and displacement curve of steel wire was not available. Therefore a strength
modification factor of 0.85 was adopted, \( f_{yw} = 0.85 f_{uw} \) (\( f_{uw} \), the average ultimate
stress of single wires). The ultimate load (\( P_2 \)), taking into account wire mesh
confinement, can be estimated using Eq. 3.4:

\[
P_2 = \left( f_{uw} + k_1 f_{yw} \right) A_c
\]

(3.4)

where the average value of the compressive strength of controls is taken as \( f_{co} \).

According to Li et al. (2001) and Martinez et al. (1984), the confinement
effectiveness, \( k_1 \), approximately equals to 4.0 for HSC confined with normal strength
steel. Li et al. (2001) suggested that for HSC confined with ultra-high yield strength,
\( k_1 \) equals to 2.7. The average ultimate strength of single lateral wires was 947 MPa,
less than the ultra-high Grade 1300 steel with yield strength of 1318 MPa (AS3600
2009). If considering wire mesh as ultrahigh yield strength steel, \( k_1 \) needs to be
adjusted to 2.7. However, the difference about the strength of confined core is
marginal due to the smallness of wire diameter. The estimated ultimate loads based
on different approaches are summarised in Table 3.8.

The approach proposed by Waliuddin and Rafeeqi (1994), as shown in Chapter 2,
was adopted to estimate the strength of confined HSC core. The equation is
presented as Eq. 3.5. It is noted that for the case of strengthening, \( K_m \) equals 0.88
(Waliuddin and Rafeeqi 1994). It is noted that the equation was based on woven
square mesh. The notations are as defined previously in Chapter 2.

\[
f_{uw} = f_{co} + 35 p K_m f_{yw}
\]

(3.5)

On the other hand, considering the confinement was not effective at the ultimate load
and taking into account of the contribution of concrete and mortar, the estimated
ultimate load (\( P_1 \)) can be computed using Eq. 3.6. It is noted that the contribution of
vertical wires can be considered negligible due to the smallness of wire diameter
(Mansur and Paramasivam 1990).

\[
P_1 = f_{co} A_c + \alpha_{t,m} f_{yw} \pi \left( D^2 - D_t^2 \right) / 4
\]

(3.6)
where $f_{co}$ is the average value of the compressive strength of controls, the coefficient $\alpha_{1,n}$ is determined in light with AS3600 (2009) as mortar is a kind of concrete, $\alpha_{1,n}$ = 1.0 - 0.003 $f_{m}^c$; $f_{m}^c$ is the mortar compressive strength at 28 days. Refer to Table 3.4 for details. The estimations based on different approaches are summarised in Table 3.8.

To evaluate the prediction accuracy, a statistic indicator of average absolute error (AAE) is adopted and determined by Eq. 3.7.

$$\text{AAE} = \frac{\sum_{i=1}^{n} |\text{mod}_i - \text{exp}_i|}{n}$$  \hspace{1cm} (3.7)

where mod = the predicted results, exp = the experimental result, and n is the total number of tested wire mesh reinforced specimens. The value of AAE for each model is presented in Table 3.8.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Test Results $P_{exp}$ (kN)</th>
<th>Predicted Results ($P_{mod}$)</th>
<th>Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$P_1$ (kN) $P_2$ (kN) $P_2$ (kN) (%) (%) (%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1L5C1</td>
<td>1676</td>
<td>1588 1486 1497</td>
<td>-5.3 -11.4 -10.7</td>
</tr>
<tr>
<td>1L5C2</td>
<td>1641</td>
<td>1588 1486 1497</td>
<td>-3.2 -9.5 -8.8</td>
</tr>
<tr>
<td>1L10C1</td>
<td>1728</td>
<td>1739 1486 1497</td>
<td>0.6 -14.0 -13.4</td>
</tr>
<tr>
<td>1L10C2</td>
<td>1517</td>
<td>1739 1486 1497</td>
<td>14.6 -2.1 -1.4</td>
</tr>
<tr>
<td>2L5C1</td>
<td>1737</td>
<td>1752 1498 1521</td>
<td>0.9 -13.7 -12.4</td>
</tr>
<tr>
<td>2L5C2</td>
<td>1713</td>
<td>1752 1498 1521</td>
<td>2.3 -12.5 -11.2</td>
</tr>
<tr>
<td>AAE (%)</td>
<td>-</td>
<td>-    -    -</td>
<td>4.5 10.5 9.6</td>
</tr>
</tbody>
</table>

Note: Deviation = $(P_{mod} - P_{exp})/P_{exp}$

As shown in Table 3.8, the estimated values of $P_2$ based on the confinement modes Eq. 3.4 and Eq. 3.5 are close. In these two confinement models, the contribution of mortar in the shell was not considered. No matter which approach was taken, the estimated values of $P_2$, except for Specimens 1L10C2, were significantly less than the experimental results. As observed in the tests, at the ultimate load the mortar cover did not spall off despite the appearance of cracking and should have contributed to the load-carrying capacity. The ultimate load was mainly sustained by the cementitious matrices of core and mortar shell; wire mesh became effective past the...
ultimate load. Thus, the contribution of mortar accordingly needs to be considered, as shown in Eq. 3.6, which provides good agreement with the test results.

As suggested by Saatcioglu and Razvi (1992), for HSC specimens, stronger confinement is required to maintain the effective ductility. This is because the relative increase in lateral strains in the inelastic range is less for HSC compared to NSC. As the expansion of concrete core is lower, the effectiveness of confinement is less (Ahmad and Shah 1982). According to Cl.10.7.3 AS3600 (2009), a minimum effective confining pressure of $0.01 f'_c$ is required for columns where $f'_c$ is greater than 50 MPa. In this experiment the amount of reinforcement ($V_{fl} = 0.19\%$, $V_{fl} = 0.38\%$) was insufficient to provide such a confining pressure and the confinement is classified as low (Cusson and Paultre 1995). However, the volume fractions were sufficient to change the sudden and brittle failure mode attributed to HSC. For HSC to achieve a second peak load, it usually requires the tie reinforcement volumetric ratio greater than 4% (Foster and Attard 2001). As a secondary confinement for HSC, wire mesh might be potentially combined with steel reinforcement to achieve a second peak load and improve the ductility significantly at a relatively low steel volumetric ratio. In this case, the number of mesh layers needs to be increased.

### 3.8 Conclusions for Experiment I

The load carrying capacity improved 10\%-17\% by using wire mesh and high strength mortar at a thin layer. The increment in the load carrying capacity tended to increase with the increase of mortar shell area. The test results suggest that at the ultimate load wire mesh confinement was not effective. The load was mainly sustained by cementitious matrices. Wire mesh become effective at the post ultimate load stage. The confinement contributed by wire mesh is limited and cannot balance the loss of cover with high compressive strength. This is reflected in the axial load and axial displacement curves. The amount of reinforcement ($V_{fl} = 0.19\%$, $V_{fl} = 0.38\%$) cannot provide the minimum effective confining pressure required by AS3600 (2009). However, the sudden and brittle failure mode changed. The ductility of wire mesh reinforced HSC specimens improved moderately at a low fraction ratio. Wire mesh influenced the post peak behaviour of confined specimens. Potentially, wire mesh might be combined with steel reinforcement to achieve a second peak load.
and improve the ductility significantly at a relatively low steel volumetric ratio for HSC.

### 3.9 Introduction of Experiment II

The second series of experiments were different from the first ones in terms of the concrete compressive strength, the cover effect and the reinforcing method. In the second series of experiments, wire mesh was used as internal reinforcement and cast with medium strength concrete (MSC), instead of strengthening existing members. Although the two applications are different in terms of the mortar shell and the interface between the wire mesh cage and the core, the effect of mesh confinement was examined in both applications. The method of forming wire mesh cage, as described in the second series of experiments, was adopted in the third series of experiments for strengthening existing concrete members, where this technique was further developed. The axial load and axial displacement behaviour of wire mesh reinforced specimens was investigated in terms of the failure mode, the load carrying capacity, ductility and energy absorption. To examine the confinement effectiveness of wire mesh separately, one layer of wire mesh was applied onto the concrete circumference as external confinement with no cover. It is notable that the specimens externally confined wire meshes behaved differently from their counterparts. Meanwhile, a detailed method of forming wire mesh reinforcement tube is also introduced.

Twelve Ø150 × 300 mm MSC specimens were cast, with a target compressive strength of 55 MPa. Wire mesh was formed into a tube with one or two layers and placed into the moulds before concrete casting. A series of tests were conducted to investigate the properties of the constituent materials. The experimental program mainly comprised specimen design, preliminary tests on properties of the constituent materials, specimen testing and results discussion. All the specimens were tested by displacement controlled concentric loading.

### 3.10 Specimen Design

The specimens were divided into six categories, as shown in Table 3.9. Each category contained two replicates. The plain concrete specimens, labelled as Control
1 and Control 2, served as reference. No reinforcement was applied. There were five categories for the wire mesh reinforced specimens. Among the alphanumeric labels of the confined specimens, L and C stand for layer and cover, respectively. The Arabic number in front of L specifies the number of mesh layers and the Arabic number in front of C indicates the nominal thickness of cover. The last Arabic number shows the number of replicates. For instance, Specimen 1L5C1 stands for the first replicate, which was confined with one layer of wire mesh and had 5 mm protective cover. All the specimens had an identical height of 300 mm.

The target concrete compressive strength was 55 MPa. A total of twelve Ø150 × 300 mm specimens were cast, among which two were made of plain concrete; eight were internally reinforced with one or two layers of wire mesh; two were externally confined with one layer of wire mesh with no cover. For the specimens integrally cast with wire mesh, the protective cover was either 5 mm or 10 mm, as shown in Fig. 3.11. The geometry of the confined concrete cores was different between each category. In Table 3.9 the diameter of the concrete core is noted as $D_c$; the overall dimension of the mean of the wire mesh layers is noted as $D_m$; the overall dimension measured between the outermost wires is noted as $D_w$; $D_m = (D_c + D_w)/2$.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$D_c$ (mm)</th>
<th>$D_m$ (mm)</th>
<th>$D_w$ (mm)</th>
<th>Cover (mm)</th>
<th>N. of Layers</th>
<th>External</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control 1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>150</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Control 2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>150</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1L5C1</td>
<td>140</td>
<td>140</td>
<td>150</td>
<td>5</td>
<td>1</td>
<td>Internal</td>
</tr>
<tr>
<td>1L10C1</td>
<td>125</td>
<td>128</td>
<td>150</td>
<td>10</td>
<td>2</td>
<td>Internal</td>
</tr>
<tr>
<td>1L5C2</td>
<td>140</td>
<td>140</td>
<td>150</td>
<td>5</td>
<td>1</td>
<td>Internal</td>
</tr>
<tr>
<td>1L10C2</td>
<td>130</td>
<td>130</td>
<td>150</td>
<td>10</td>
<td>2</td>
<td>Internal</td>
</tr>
<tr>
<td>2L5C1</td>
<td>135</td>
<td>138</td>
<td>150</td>
<td>5</td>
<td>2</td>
<td>Internal</td>
</tr>
<tr>
<td>2L5C2</td>
<td>135</td>
<td>138</td>
<td>150</td>
<td>5</td>
<td>2</td>
<td>Internal</td>
</tr>
<tr>
<td>2L10C1</td>
<td>125</td>
<td>128</td>
<td>150</td>
<td>10</td>
<td>2</td>
<td>Internal</td>
</tr>
<tr>
<td>2L10C2</td>
<td>125</td>
<td>128</td>
<td>150</td>
<td>10</td>
<td>2</td>
<td>Internal</td>
</tr>
<tr>
<td>1L0C1</td>
<td>150</td>
<td>150</td>
<td>152</td>
<td>-</td>
<td>1</td>
<td>External</td>
</tr>
<tr>
<td>1L0C2</td>
<td>150</td>
<td>150</td>
<td>152</td>
<td>-</td>
<td>1</td>
<td>External</td>
</tr>
</tbody>
</table>

Note:* Diameter of the concrete core;
\[ \text{The overall dimension of the mean of the wire mesh layers; equal to } D_c + (D_w - D_c)/2 = (D_c + D_w)/2 \]
\[ \text{The overall dimension measured between the outermost wires} \]
3.11 Wire Mesh Property

Locally available 12.7 mm×12.7 mm galvanised weld wire mesh, having a nominal diameter of 1.2 mm, was used in this study. To investigate tensile strength of wire mesh, the tests were conducted on 280 mm long single-wire samples in both transverse and longitudinal directions. In addition, 5-strand mesh coupons and 7-strand mesh coupons from the transverse direction were also prepared and tested.

3.11.1 Ultimate Stress of Single Wires

The direct tensile strength tests of single wire were conducted using Testometric, M500-50 kN at a speed of 0.3 mm/min. The measured average diameter of single wires in transverse direction was 1.1 mm. The average ultimate stress in the transverse and longitudinal direction was 565 MPa and 545 MPa, respectively. Lateral wires and vertical wires had similar ultimate stress since the average ultimate...
stress of lateral was 3.67% higher than that of vertical wires. It is reasonable to consider specimens were reinforced uniformly in both orientations.

3.11.2 Wire Mesh Coupons

The 5-strand mesh coupons and 7-strand mesh coupons were indicated as 5SMC and 7SMC, respectively. Mesh coupons were cut from the same roll of wire mesh in the transverse direction. The ends of each coupon were embedded in mortar. The preparation of mesh coupons was in accordance with ACI549.1R-1999. The dimension of 5-strand mesh coupons and 7-strand mesh coupons was 250 mm × 50 mm and 390 mm × 78 mm, respectively; the free length for 5-strand mesh coupons and 7-strand mesh coupons was 150 mm and 234 mm, respectively. The thickness of mortar ends was 25 mm.

The direct tensile strength tests were carried out using MTS793 under displacement control at a speed of 1 mm/min. The tests of the tensile strength for all the mesh coupons were conducted in the Mechanics Laboratory at the University of Hong Kong. The load and elongation curves of 5-strand mesh coupons and 7-strand mesh coupons are illustrated in Fig. 3.12 and Fig. 3.13, respectively. As can be seen from the two figures, a sudden drop in load appeared due to the snapping of the wires in the axial direction. In general, wire stands failed one by one randomly after passing the ultimate load. This is possibly due to the difference in geometry and material property of single wires (Arif and Kaushik 1999). In some cases wire strands ruptured simultaneously beyond the ultimate tensile load. These two patterns appeared in the tested confined specimens.
Fig. 3.12 Tensile strength test of 5-strand mesh coupon specimens

Fig. 3.13 Tensile strength test of 7-strand mesh coupon specimens

As illustrated in the two figures, there was no clear distinguishing point between the yield load and the ultimate load. The tensile strength test results of the mesh coupons are summarised in Table 3.10, which indicates that the dimension difference did not significantly influence the strength since the average value of the ultimate strength of the two types of coupons was the same, 557 MPa. Arbitrary yield strength was obtained using the offset method, 0.2% standard strain. Again, the yield strength was almost the same. The results of the direct tensile strength show that the ultimate
stress of mesh coupons and single wires were close. The average yield stress of 5-strand mesh coupons was adopted for determining the confinement pressure.

Table 3.10 Tensile strength of wire mesh coupons

<table>
<thead>
<tr>
<th>Mesh Coupon</th>
<th>Dimension mm</th>
<th>Yield Stress (MPa)</th>
<th>Ultimate Stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean Coe.* Variation</td>
<td>Mean Coe.* Variation</td>
</tr>
<tr>
<td>5-Strand</td>
<td>50×250</td>
<td>526.6 0.026</td>
<td>557.2 0.041</td>
</tr>
<tr>
<td>7-Strand</td>
<td>78×390</td>
<td>529.4 0.038</td>
<td>557.2 0.037</td>
</tr>
</tbody>
</table>

Note: Coe. * = coefficient

3.12 Forming Wire Mesh Reinforcement Tube

As internal confinement, wire mesh was formed into a tube with one or two layers and placed into the steel moulds before concrete casting. Based on calculation, right lengths of wire mesh were cut off from the same roll, which allowed forming concrete cover on the top and bottom. Hose clips and ply timber plates with a smoothed circular opening of 140 mm and 130 mm in diameter, respectively, were used to form a mesh cage, as shown in Fig. 3.14. The tube was approximately 280 mm in height. The overlap area was approximately 76 mm wide, which was approximately equal to 2/5 of the perimeter of the confined core. To ensure that the overlap would not loosen or slide from each other, it was tied with steel wire with similar diameter at each side of the overlap at a spacing of every two grids (approximately 25 mm). Additional ties were applied to the middle of the overlap area at a 76 mm spacing (6 grids), approximately. Steel wires were used to allow the space around the reinforcement tube for 5 mm or 10 mm cover, respectively. A number of vertical steel wires were kept symmetrically around the bottom of the reinforcement cage to allow 10 mm concrete cover, approximately.
To investigate how wire mesh affects the behaviour of concrete specimens, two Ø150 × 300 mm plain MSC specimens were externally confined with one layer of wire mesh only and no cover was applied. As external confinement, wire mesh was wrapped to the concrete circumference in a similar way after 28 days since concrete casting. It is noted that using wire mesh as external confinement with no protection cover is probably not appropriate for the severe environmental conditions. However, as a means of investigation, it enables to examine the confinement effectiveness of wire mesh separately.

3.13 Medium Strength Concrete (MSC)

In addition to the twelve Ø150 × 300 mm specimens, seven Ø100 × 200 mm cylinders were cast and tested for the specified concrete properties. For the specimens internally reinforced with wire mesh, mesh tubes were centralised in the steel moulds before concrete casting. Refer to Section 3.2 for concrete mix, cast and curing. The concrete had a slump of 150 mm.

The tests of the concrete compressive strength tests were conducted on Ø100×200mm cylinders using DATAMATIC, which had a capacity of 2000 kN, in the Building Materials Laboratory at Hong Kong Polytechnic University. To ensure a
smooth surface, each end of all the seven Ø100 × 200 mm cylinders was capped with sulphur before testing. The test speed was manually controlled. The average compressive strength at 28 days was 54.5 MPa.

To investigate concrete modulus of elasticity, three standard cylinders (Ø100 × 200 mm) were tested in accordance to BS 1881-121:1983. The modulus of elasticity test was conducted at Day 29, as shown in Fig. 3.15. The results of modulus of elasticity are presented in Table 3.11. The experimental results were more close to the values suggested by ACI 318M-08 than AS3600 (2009). The average of the experiment results was adopted for analysing the stress and strain relationship of Specimen 1L0C1. Refer to Section 3.17.3 for details. At the end of the modulus elasticity test, the average compressive strength of the same samples was 56.9 MPa.

![Fig.3.15 Young Modulus of Elasticity Test](image)

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Strain Gauge Length (mm)</th>
<th>Experiment $E_c$, Exp (MPa)</th>
<th>ACI318* $E_c$, ACI318 (MPa)</th>
<th>Deviation $[(4)-(3)]/(3)$ (5) %</th>
<th>AS3600 ** $E_c$, AS3600 (MPa)</th>
<th>Deviation $[(6)-(3)]/(3)$ (7) %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100</td>
<td>34048</td>
<td>35453</td>
<td>4.13</td>
<td>35996</td>
<td>5.72</td>
</tr>
<tr>
<td>2</td>
<td>100</td>
<td>33105</td>
<td>35453</td>
<td>7.09</td>
<td>35996</td>
<td>8.73</td>
</tr>
<tr>
<td>3</td>
<td>100</td>
<td>33451</td>
<td>35453</td>
<td>5.98</td>
<td>35996</td>
<td>7.61</td>
</tr>
<tr>
<td>4</td>
<td>100</td>
<td>34458</td>
<td>35453</td>
<td>2.89</td>
<td>35996</td>
<td>4.46</td>
</tr>
</tbody>
</table>

Note: * ACI 318M-08; ** AS3600 (2009)
3.14 Preparation of Final Specimens

The ends of each specimen were strengthened and capped in a similar way as described in the previous Section 3.6. Three strain gauges with a gauge length of 100 mm were vertically and equally placed around the circumference at the middle height. Three strain gauges with a gauge length of 50 mm were placed equally around the circumference at the middle height. Refer to Fig. 3.16 for details. The strain gauges were provided by the same manufacturer and applied following the same procedure introduced in Section 3.6.

Two Ø150 × 300 mm plain MSC cylinders, Marked as 1L0C1 and 1L0C2, were externally confined with one layer of wire mesh only, as shown in Fig. 3.17. The wire mesh edge is below and above the ends, which avoided the situation that load could be carried by vertical wires. Hose clips were used to facilitate wrapping wire mesh onto the concrete circumference closely. Specimens1L0C1 and 1L0C2 were not roughened. Strain gauges were installed at the same locations as for internally reinforced specimens. Coating glue was used to protect strain gauges, which caused a small gap between the mesh cage and the concrete core in some small areas. Epoxy (HM-E8 Heroman) coating glue and mortar were used to fill up the gap only. An approximately 40 mm wide wire mesh strip was used to strengthen each end.
Testing Setup and Failure Modes

3.15.1 Testing Setup

Two Controls and the mesh-confined specimens were tested using MTS 643, which has a capacity of 4600 kN. The test was conducted under the displacement control at a speed of 0.15 mm/min (strain rate 0.0025 mm/s). All the specimens were tested under concentric loading. Two Linear Variable Differential Transformers (LVDTs) were placed opposite to each other on a steel frame, which was firmly attached to the specimen. The gauge length of each LVDT was 120 mm. All the specimens were tested to failure. Before testing, the machine was calibrated and each specimen was adjusted to the centre.

3.15.2 Failure Modes

In general, for all the confined specimens, cover peeled or unloaded in a gradual manner. Compared to their counter parts in Experiment I, the failure modes exhibited more ductility. For the specimens internally reinforced with wire mesh, fine vertical cracks initially appeared. Cracks continued extending in length and increasing in width. Large vertical cracks developed almost throughout the height of specimens while lateral cracks also appeared. However, vertical cracking was the dominated phenomenon. It is noted that scale pattern also appeared in Specimen 1L10C1. It was
often found that vertical wires buckled near the CFRP stripes due to stress concentration. The edges along the overlap were the weak point where lateral wires ruptured. The wires used to tie up the mesh layers at the overlap area remained intact.

In-plane shear failure and potential double core failure were presented in the core of Specimens 1L5C1 and 1L5C2, respectively. Lateral wires ruptured and vertical wires buckled along the patterns. Typical vertical crack with approximately half length of the full height appeared on the core of Specimen 1L5C2, evidencing that wire mesh cage was in tension. In-plane shear failure was observed in the core of Specimen 1L10C2 oriented at an angle value with the horizontal of 70 degrees, approximately. The inclination angle was close to the estimation proposed by Cusson and Paultre (1995). Meanwhile, vertical cracking with a length of more than one third of full height appeared along one side of overlap area. As indicated by Mander et al. (1988 b), a clearly diagonal failure plane indicates low volumetric ratio of steel confinement for concrete columns.

Potential double core failure and in-plane shear failure were exhibited in the core of Specimens 2L5C1 and 2L5C2, respectively. Similar to Specimens 1L5C1 and 1L5C2, lateral wires ruptured and vertical wires buckled at the interstices along the pattern. It was also observed that vertical wires buckled at the interstices around the confined core. Potential double core failure appeared in Specimen 2L10C1; in-plane shear failure oriented at an angle value with the horizontal of 73 degrees, approximately, was observed in Specimen 2L10C2. Selected tested specimens are shown in Fig. 3.18.

For Specimens 1L0C1 and 1L0C2, it was observed: the specimens bulged at the middle and the concrete core seemed crashed. Wire mesh tube deformed significantly. Lateral wires ruptured along vertical cracks. Some lateral wires ruptured at the same vertical position. Vertical wires mainly buckled around the circumference close to the middle height. Lateral wires were also broken along the edge of the overlap. Although the specimens bulged, the wire ties remained intact, as shown in Fig. 3.17 (b).
3.16 Test Results and Discussion (Internally Reinforced Specimens)

3.16.1 Axial Load and Axial Displacement

The test results are summarised in Table 3.12, while the axial load and axial displacement curves are illustrated in Fig. 3.19 and Fig. 3.20.
## Table 3.12 Experiment II test results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Volumetric Ratio</th>
<th>Volume Fraction</th>
<th>Yield Load ( P_y )</th>
<th>Ultimate Load ( P_u )</th>
<th>Average Load ( P_u ) at ( P_u )</th>
<th>Axial Displacement ( \Delta u ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control 1</td>
<td>-</td>
<td>-</td>
<td>866 (kN)</td>
<td>924 (kN)</td>
<td>1.330</td>
<td></td>
</tr>
<tr>
<td>Control 2</td>
<td>-</td>
<td>-</td>
<td>778 (kN)</td>
<td>829 (kN)</td>
<td>877 (kN)</td>
<td>1.150 (kN)</td>
</tr>
<tr>
<td>1L5C1</td>
<td>0.214</td>
<td>1.496</td>
<td>713 (kN)</td>
<td>823 (kN)</td>
<td>1.580</td>
<td></td>
</tr>
<tr>
<td>1L5C2</td>
<td>0.214</td>
<td>1.496</td>
<td>742 (kN)</td>
<td>843 (kN)</td>
<td>833 (kN)</td>
<td>1.500 (kN)</td>
</tr>
<tr>
<td>1L10C1</td>
<td>0.230</td>
<td>0.748</td>
<td>807 (kN)</td>
<td>909 (kN)</td>
<td>1.560</td>
<td></td>
</tr>
<tr>
<td>1L10C2</td>
<td>0.230</td>
<td>0.748</td>
<td>754 (kN)</td>
<td>852 (kN)</td>
<td>881 (kN)</td>
<td>1.400 (kN)</td>
</tr>
<tr>
<td>2L5C1</td>
<td>0.420</td>
<td>1.994</td>
<td>762 (kN)</td>
<td>862 (kN)</td>
<td>1.650</td>
<td></td>
</tr>
<tr>
<td>2L5C2</td>
<td>0.420</td>
<td>1.994</td>
<td>799 (kN)</td>
<td>917 (kN)</td>
<td>890 (kN)</td>
<td>1.850 (kN)</td>
</tr>
<tr>
<td>2L10C1</td>
<td>0.451</td>
<td>1.197</td>
<td>745 (kN)</td>
<td>840 (kN)</td>
<td>1.430</td>
<td></td>
</tr>
<tr>
<td>2L10C2</td>
<td>0.451</td>
<td>1.197</td>
<td>768 (kN)</td>
<td>881 (kN)</td>
<td>861 (kN)</td>
<td>1.590 (kN)</td>
</tr>
<tr>
<td>1L0C1</td>
<td>0.199</td>
<td>-</td>
<td>988 (kN)</td>
<td>1083 (kN)</td>
<td>1.490</td>
<td></td>
</tr>
<tr>
<td>1L0C2</td>
<td>0.199</td>
<td>-</td>
<td>967 (kN)</td>
<td>1032 (kN)</td>
<td>1058 (kN)</td>
<td>1.390 (kN)</td>
</tr>
</tbody>
</table>

Note: *\( \rho_w \): lateral wires with respect to the core; *\( V_{fl} \): lateral wires with respect to the shell.

![Fig. 3.19 Axial load- axial displacement of internally reinforced specimens](image)

Fig. 3.19 Axial load- axial displacement of internally reinforced specimens
The results indicate that wire mesh reinforced specimens did not outperform the controls in terms of the average ultimate load, but in terms of ductility. The controls demonstrated a brittle behaviour. By contrast, wire mesh reinforced specimens exhibited a relatively high ductility evidenced by a gradual drop in the descending branch of the curves. The investigation on the tested specimens showed that lateral wires ruptured in tension, as observed in the first series of experiments. Unlike the specimens sufficiently confined with steel spirals or hoops, there was no second peak load or load plateau appeared, except for Specimen 2L5C2. The axial load and displacement curve of Specimen 2L5C2 suggests this specimen might be effective at the ultimate load. For the specimens reinforced with two layers of wire mesh, Specimens 2L5C1 and 2L5C2 outweighed Specimens 2L10C1 and 2L10C2 in terms of the ultimate load, which indicates a less thick cover might be beneficial. For Specimens confined with one layer of wire mesh, Specimens 1L10C1 and 1L10C2 had higher value in the ultimate load than Specimens 1L5C1 and 1L5C2. However, Specimens 1L5C1 and 1L5C2 presented a more smooth turn between the ascending and descending proportion.
For the specimens cast integrally with wire mesh cage, the increment in the number of mesh layers did not accordingly result in an increase in the ultimate load. This observation is contrast to what was commonly reported in the literature about applying wire mesh in normal strength concrete members (Waliuddin and Rafeeqi, 1994). This phenomenon is probably due to: 1) the integration between the core and the cover was weakened due to the presence of wire mesh with small aperture size; 2) the gain in the load carrying capacity attributed to wire mesh reinforcement is small compared to the load carried by the concrete cover; and 3) the concrete had relatively high compressive strength.

On the other hand, in the experimental work conducted by Chen (2010), it was found that applying one layer of wire mesh did not increase the ultimate load for the cylindrical specimens with 10 mm cover, increased 2% for the specimens with 5 mm cover. The materials properties in Chen’s study are: the specified concrete compressive strength was 48 MPa, based on 100 mm cubic samples at 28 days; the wire mesh had a diameter of 1.4 mm, opening size of 10 mm × 10 mm; the yield strength of single wire was 748 MPa. All the cylindrical specimens had an identical diameter of 150 mm, while the height varied from 2 to 3 times of the diameter.

3.16.2 Ductility

The same two ductility ratios, \( \mu \) and \( \mu_{s5} \), were adopted. Refer to the previous Section 3.7.2 for the definition of these two ratios. The results of ductility are presented in Table 3.13 and illustrated in Fig. 3.21, which indicates that the two ductility ratios considerably increased. Ductility ratio of \( \mu_{s5} \) improved more significantly for the specimens internally confined with wire mesh.
Table 3.13 Ductility of Experiment II specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield Load $P_y$ (kN)</th>
<th>Ultimate Load $P_u$ (kN)</th>
<th>Axial Displacement</th>
<th>Ductility Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\Delta u$ (mm)</td>
<td>$\Delta y$ (mm)</td>
<td>$\Delta_{85}$ (mm)</td>
<td>$\mu_y$</td>
</tr>
<tr>
<td>Control 1</td>
<td>866</td>
<td>924</td>
<td>1.330</td>
<td>1.18</td>
</tr>
<tr>
<td>Control 2</td>
<td>778</td>
<td>829</td>
<td>1.150</td>
<td>1.15</td>
</tr>
<tr>
<td>1L5C1</td>
<td>713</td>
<td>823</td>
<td>1.580</td>
<td>1.56</td>
</tr>
<tr>
<td>1L5C2</td>
<td>742</td>
<td>843</td>
<td>1.500</td>
<td>1.42</td>
</tr>
<tr>
<td>1L10C1</td>
<td>807</td>
<td>909</td>
<td>1.560</td>
<td>1.43</td>
</tr>
<tr>
<td>1L10C2</td>
<td>754</td>
<td>852</td>
<td>1.400</td>
<td>1.35</td>
</tr>
<tr>
<td>2L5C1</td>
<td>762</td>
<td>862</td>
<td>1.650</td>
<td>1.36</td>
</tr>
<tr>
<td>2L5C2</td>
<td>799</td>
<td>917</td>
<td>1.850</td>
<td>1.64</td>
</tr>
<tr>
<td>2L10C1</td>
<td>745</td>
<td>840</td>
<td>1.430</td>
<td>1.38</td>
</tr>
<tr>
<td>2L10C2</td>
<td>768</td>
<td>881</td>
<td>1.590</td>
<td>1.45</td>
</tr>
<tr>
<td>1L0C1</td>
<td>988</td>
<td>1083</td>
<td>1.490</td>
<td>1.26</td>
</tr>
<tr>
<td>1L0C2</td>
<td>967</td>
<td>1032</td>
<td>1.390</td>
<td>1.19</td>
</tr>
</tbody>
</table>

Fig. 3.21 Ductility of Experiment II specimens

3.16.3 Estimation for the Ultimate Load of Internally Reinforced Specimens

As indicated in the previous Section 3.7.3, it is reasonable to consider lateral wire strands as closely spaced circular steel spirals with small diameter. Eq. 3.4 and Eq. 3.5 were adopted to estimate the ultimate load due to wire mesh confinement. It is
noted that the requirement about the minimum effective confining pressure (AS3600 2009) was satisfied. For Eq. 3.4, the confinement effectiveness, $k_1$ equals to 4.0 (Martinez et al. 1984); for Eq. 3.5, $K_m$ equals 1 for the case of concrete cast with mesh reinforcement (Waliuddin and Rafeeqi 1994). On the other hand, considering wire mesh not effective at the ultimate load, the load carrying capacity ($P_1$) can be calculated using the following equation:

$$P_1 = f_{co}A_c + \alpha_1 f_{co} \pi \left(D^2 - D^*_e^2\right) / 4$$  \hspace{1cm} (3.8)

where $f_{co}$ is the in-situ compressive strength of unconfined specimen; $\alpha_1 = 1.0 - 0.003 f_{co}$ based on AS 3600 (2009); $A_c$ is as defined previously. The strength of concrete cover is reduced due to small aperture size of wire mesh, which resulted in coarse aggregates separated from the cover shell. The estimations based on different approaches are summarised in Table 3.14.

| Specimen  | Test Results $P_{exp}$ (kN) | Predicted Results $P_{mod}$ ($P_{eq}$) | $\%$ | $\%$ | $\%$
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Eq. 3.8</td>
<td>Eq. 3.4</td>
<td>Eq. 3.5</td>
<td>Eq. 3.8</td>
</tr>
<tr>
<td>1L5C1</td>
<td>823</td>
<td>860</td>
<td>795</td>
<td>832</td>
<td>4.5</td>
</tr>
<tr>
<td>1L5C2</td>
<td>843</td>
<td>860</td>
<td>795</td>
<td>832</td>
<td>2.0</td>
</tr>
<tr>
<td>1L10C1</td>
<td>909</td>
<td>844</td>
<td>687</td>
<td>715</td>
<td>-7.1</td>
</tr>
<tr>
<td>1L10C2</td>
<td>852</td>
<td>844</td>
<td>687</td>
<td>715</td>
<td>-1.0</td>
</tr>
<tr>
<td>2L5C1</td>
<td>862</td>
<td>860</td>
<td>828</td>
<td>898</td>
<td>-0.2</td>
</tr>
<tr>
<td>2L5C2</td>
<td>917</td>
<td>860</td>
<td>828</td>
<td>898</td>
<td>-6.3</td>
</tr>
<tr>
<td>2L10C1</td>
<td>840</td>
<td>844</td>
<td>718</td>
<td>770</td>
<td>0.5</td>
</tr>
<tr>
<td>2L10C2</td>
<td>881</td>
<td>844</td>
<td>718</td>
<td>770</td>
<td>-4.2</td>
</tr>
<tr>
<td>AAE (%)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>3.2</td>
</tr>
</tbody>
</table>

Note: $*P_1$: the load capacity contributed by concrete; $\%P_2$: the load capacity when wire mesh reinforcement became effective. Deviation=$\left(P_{mod} - P_{exp}\right) / P_{exp}$

As shown in Table 3.14, the variation between the estimated and the experimental results for Eq. 3.4 and Eq. 3.5 was about the specimens reinforced with one layer of wire mesh and having a protective cover of 10 mm. For the same cover thickness, the variation became less for specimens 2L10C1 and 2L10C2. For the specimens with 5 mm cover, the variation is significant less than their counterparts with 10 mm cover.
The results suggested that at the ultimate load, for most of the specimens, wire mesh was likely not effective and concrete cover sustained some load. If considering wire mesh confinement effective and accordingly only taking the area of core into account, the estimated value is considerably less than experiment results. On the other hand, considering wire mesh not effective at the ultimate load and taking into account of the contribution of concrete cover, Eq. 3.8 provides estimation with reasonable accuracy. The estimated values compared with the experimental results are illustrated in Fig. 3.22.

The results of Chen’s study (2010) presented in Table 3.15. The results were based on the average value of three replicates. It is noted that the estimated values of $P_1$ for the specimens in Table 3.15 are also included in Fig. 3.22 (a).

### Table 3.15 Data from Chen’s experiment (2010)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Diameter (mm)</th>
<th>Height (mm)</th>
<th>Cover (mm)</th>
<th>No. of mesh Layers</th>
<th>Ultimate load $P_u$ (kN)</th>
<th>Estimation $P_1$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>150</td>
<td>300</td>
<td>-</td>
<td>-</td>
<td>627</td>
<td>-</td>
</tr>
<tr>
<td>1L10C300H</td>
<td>150</td>
<td>300</td>
<td>10</td>
<td>1</td>
<td>627</td>
<td>610</td>
</tr>
<tr>
<td>1L10C375H</td>
<td>150</td>
<td>375</td>
<td>10</td>
<td>1</td>
<td>605</td>
<td>610</td>
</tr>
<tr>
<td>1L10C450H</td>
<td>150</td>
<td>450</td>
<td>10</td>
<td>1</td>
<td>581</td>
<td>610</td>
</tr>
<tr>
<td>1L5C300H</td>
<td>150</td>
<td>300</td>
<td>5</td>
<td>1</td>
<td>640</td>
<td>618</td>
</tr>
</tbody>
</table>

![Fig. 3.22 Results comparison](image-url)
For concrete with relative high strength and cast integrally with wire mesh, whether wire mesh is effective at the ultimate load is sensible to the cover thickness. The axial load and axial displacement curves provide some indication on whether wire mesh was effective at the ultimate load point. The proposed Eq. 3.8 provides close estimation for all the wire mesh confined specimens of this study, particularly for the case of wire mesh not effective at the ultimate load stage.

### 3.17 Test Results and Discussion (Externally Reinforced Specimens)

3.17.1 Axial load and axial displacement curves

As mentioned earlier, Specimens 1L0C1 and 1L0C2 were externally wrapped with 1 layer of wire mesh with no cover. The axial load and axial displacement curves of the two specimens are illustrated in Fig. 3.23. It is interesting to see Specimens 1L0C1 and 1L0C2 behaved differently from their counterparts, the specimens internally reinforced with wire mesh. Specimens 1L0C1 and 1L0C2 clearly outperformed their counterparts in terms of the ultimate load, as shown in Table 3.13. However, the increment in ductility was modest.

![Fig. 3.23 Axial load- axial displacement of externally confined specimens](image)
3.17.2 Axial load and axial strain curves

To investigate the axial strain at the ultimate load, the axial load and axial strain curves are plotted, as shown in Fig. 3.24. The axial load and axial strain curves were generated by dividing the average LVDT reading by the gauge length. Due to cracking and for the convenience of comparison, all the axial load and axial strain curves were discontinued at the ultimate load. It is also noted that it is not possible to determine the full axial load and axial strain curves due to the inherent limitations associated with the instrumentation. Fig. 3.24 shows that at the ultimate load the axial strain of wire mesh reinforced specimens was significantly larger than that of the controls, which suggests that significant axial deformation occurred.

![Axial load-axial strain of externally reinforced specimens](image)

Fig. 3.24 Axial load-axial strain of externally reinforced specimens

3.17.3 Estimation for the Ultimate Load of Externally Reinforced MSC Specimens

The in-fill material might have limited contribution to the load carrying capacity. However, due to the nature and very small quantity, the contribution of in-fill materials was considered negligible. Three approaches were adopted to estimate the ultimate load: 1) based on Eq. 3.4; 2) steel tube considering the equivalent continuous jacket thickness; and 3) the method proposed by Kaish et al. (2015).
The second approach was involved with the concept of equivalent continuous jacket thickness, taking into account lateral wires only. In this study, wires were evenly and closely spaced in both longitudinal and transverse directions. Therefore, the external reinforcing wire mesh tube can be considered as a kind of steel tube. The second is discussed in the following section. The estimated ultimate loads \((P_2)\) based on the three approaches are summarised in Table 3.17.

In light with the equivalent continuous jacket thickness for steel reinforcement (Binici 2005), the equivalent continuous jacket thickness for wire mesh can be calculated using Eq. 3.9.

\[
t_{jw} = \frac{n A_w}{s_w} \left( 1 - \frac{s_w}{\sqrt{1.25D_c}} \right)
\]  

where \(t_{jw}\) is the thickness of the equivalent continuous jacket; \(n, A_w\) and \(s_w\) are as defined previously; \(D_c\) is the diameter of the confined concrete. When the tube thickness is most significantly less than the diameter of confined concrete \((t_{jw} \ll D)\), the reinforcement ratio, \(\mu_{sw}\), can be determined using Eq. 3.10 (Guo and Shi 2003).

\[
\mu_{sw} = \frac{4t_{jw}}{D_c}
\]  

(3.10)

The confinement ratio is defined as Eq. 3.11 and the ultimate load is calculated using Eq. 3.12.

\[
\lambda_{sw} = \frac{4t_{jw}f_{yw}}{D_c f_{co}} = \mu_{sw} \frac{f_{yw}}{f_{co}}
\]  

(3.11)

\[
P_2 = f_{co} \left( 1 + 2\lambda_{sw} \right) A_c
\]  

(3.12)

The method proposed by Kaish et al. (2015), shown as Eq. 2.55 and Eq. 2.56, was adopted. As mentioned in Chapter 2, this method tends to overestimate for the case of mortar shell. However, for the case of external confinement with no cover, this method provides relatively close estimation.
As can be seen from Table 3.16, the estimation based on equivalent steel tube was 14.5% less than the experimental results. This is probably because with the presence of closely spaced vertical wires, it will tend to underestimate the ultimate load if only taking into account lateral wires for the equivalent thickness. Table 3.16 indicates that Specimens 1L0C1 and 1L0C2 had significant increment in the ultimate load. Due to lack of data, further investigation is recommended to provide a more accurate model for the case of no cover applied.

### Table 3.16 Results comparison for externally confined specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Test Results (kN)</th>
<th>Predicted Results ($P_{\text{mod}}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Eq. 3.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$P_1$ (kN)</td>
</tr>
<tr>
<td>1L0C1</td>
<td>1082.6</td>
<td>910.6</td>
</tr>
<tr>
<td>1L0C2</td>
<td>1032.4</td>
<td>910.6</td>
</tr>
<tr>
<td>Deviation</td>
<td>-</td>
<td>-13.9%</td>
</tr>
</tbody>
</table>

Note: *Kaish et al. (2015)*  
Deviation = ($P_{\text{mod}}$ - $P_{\text{exp}}$) / $P_{\text{exp}}$ (%); $P_{\text{exp}}$ = Average test results; Equit. = equivalent.

Based on the method of Kaish et al. (2015), the estimated strength of confined concrete was 55.0 MPa. The equations Eq. 2.16-Eq. 2.19 (Chapter 2) proposed by Mander et al. (1988 a) were adopted for analysing the stress and strain relationship of externally confined concrete. The average experiment results of concrete modulus of elasticity 33.8 GPa was used for determining the constant $r$. The analytical and experimental axial stress and axial strain curve of Specimen 1L0C1 are plotted in Fig. 3.25, which shows the analytical values were less than the experimental results within a reasonable range. For Specimen 1L0C2 the variation between this analytical model and the experimental results was large and thus excluded from the plot.
3.18 Energy Absorption

As mentioned in Section 3.7, AUC indicates the energy absorption capacity (Ganesan and Anil 1993). The same index was used for comparing the energy absorption capability. The values of $I_{50}$ for each specimen are shown in Table 3.17. This table indicates that wire mesh has advantage in energy absorption and the energy absorption index is mainly affected by the number of mesh layers (fraction volume). For the specimens with the same layer of wire mesh internally, the effect of cover thickness on the energy absorption capacity was marginal.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Area under the axial load and deflection curve (AUC), kN.mm</th>
<th>Energy absorption Index, $I_{50}$</th>
<th>Average $I_{50}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control 1</td>
<td>894</td>
<td>1*</td>
<td>-</td>
</tr>
<tr>
<td>Control 2</td>
<td>789</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1L5C1</td>
<td>1441</td>
<td>1.70</td>
<td></td>
</tr>
<tr>
<td>1L5C2</td>
<td>1822</td>
<td>2.15</td>
<td>1.93</td>
</tr>
<tr>
<td>1L10C1</td>
<td>1790</td>
<td>2.12</td>
<td></td>
</tr>
<tr>
<td>1L10C2</td>
<td>1497</td>
<td>1.77</td>
<td>1.94</td>
</tr>
<tr>
<td>2L5C1</td>
<td>1811</td>
<td>2.14</td>
<td></td>
</tr>
<tr>
<td>2L5C2</td>
<td>1910</td>
<td>2.26</td>
<td>2.20</td>
</tr>
<tr>
<td>2L10C1</td>
<td>1847</td>
<td>2.18</td>
<td></td>
</tr>
<tr>
<td>2L10C2</td>
<td>1966</td>
<td>2.32</td>
<td>2.25</td>
</tr>
<tr>
<td>1L0C1</td>
<td>1622</td>
<td>1.92</td>
<td></td>
</tr>
<tr>
<td>1L0C2</td>
<td>1232</td>
<td>1.46</td>
<td>1.69</td>
</tr>
</tbody>
</table>

Note: *Take the average value $I_{50}$ as the reference.
3.19 Conclusions for Experiment II

With the amount of reinforcement ($V_r$ ranged from 0.75% to 1.99%), the requirement about the minimum effective confining pressure (AS3600 2009) was satisfied. For the specimens internally reinforced with wire mesh, the ultimate load increased marginally, but the ductility increased significantly. In other words, the effect of wire mesh on ductility was pronounced than on the ultimate load. The increment in the layer of wire mesh did not ensure the increase in the ultimate load, which is contrast to what was commonly reported in the literature about applying wire mesh in NSC cylindrical specimens. This is due to the gain in the strength of confined core could not exceed the load carried by concrete cover. Due to the nature of reinforcing mesh, whether wire mesh is effective at the ultimate load is sensitive to the cover thickness. Eq. 3.8 provides close estimation for all the wire mesh confined specimens of this experiment, particularly for the case of wire mesh not effective at the ultimate load stage. The specimens externally confined with wire mesh and no cover applied behaved differently. The ultimate load increased significantly and ductility improved modest. Again, wire mesh exhibited good capacity in energy absorption.
CHAPTER 4  TRIAL TEST FOR EXPERIMENT III

4.1 Introduction

To examine the method of using steel hoop, wire mesh and mortar to strengthen concrete, a trial test was carried out in the High Bay Laboratory at the University of Wollongong. This chapter mainly covers the specimen preparation, the properties of the constituent materials, the testing of confined specimens and test results.

4.2 Materials Properties and Preparation for Trial Specimens

Three relatively large cylindrical specimens, $\Omega 150 \times 380$ mm, were cast and confined with different amount of wire mesh and steel hoops. The reparation mainly comprised three steps: concrete core casting, reinforcement installation and mortar application.

4.2.1 Concrete Mixes

Concrete used for the trial test was mixed in the laboratory. The target compressive strength was 50 MPa. In total, twenty one standard $\Omega 100 \times 200$ mm cylinders and three $\Omega 150 \times 380$ mm cylinders were cast in three batches. The latter were $\Omega 150 \times 380$ mm specimens were cast within the first batch. The moulds for $\Omega 150 \times 380$ mm specimens were made out of PVC pips with an inner diameter of 150 mm, while standard steel moulds were used for casting $\Omega 100 \times 200$ mm cylinders.

The concrete mixture proportions are presented in Table 4.1. General purpose cement (GP Type), nature river sand and crushed graded coarse aggregate were used for the concrete mix. Before casting, water content for each type of aggregate was tested. A motor-driven mixer was used for the mixing. Refer to Section 3.2 for the concrete mixing and casting.
Table 4.1 Concrete mixture proportion

<table>
<thead>
<tr>
<th>Material</th>
<th>Batch (kg)</th>
<th>I</th>
<th>II</th>
<th>III</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water</td>
<td></td>
<td>7.43</td>
<td>2.50</td>
<td>2.69</td>
</tr>
<tr>
<td>Cement</td>
<td></td>
<td>13.60</td>
<td>4.90</td>
<td>3.98</td>
</tr>
<tr>
<td>Fine sand</td>
<td></td>
<td>10.91</td>
<td>3.32</td>
<td>3.75</td>
</tr>
<tr>
<td>Coarse sand</td>
<td></td>
<td>21.95</td>
<td>6.91</td>
<td>7.35</td>
</tr>
<tr>
<td>Agg. 10 mm</td>
<td></td>
<td>11.00</td>
<td>3.57</td>
<td>3.55</td>
</tr>
<tr>
<td>Agg. 20 mm</td>
<td></td>
<td>28.21</td>
<td>9.16</td>
<td>9.16</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td></td>
<td>0.272</td>
<td>0.096</td>
<td>0.09</td>
</tr>
<tr>
<td>Volume (m³)</td>
<td></td>
<td>0.0389</td>
<td>0.0126</td>
<td>0.0126</td>
</tr>
<tr>
<td>W/C ratio</td>
<td></td>
<td>0.55</td>
<td>0.51</td>
<td>0.68</td>
</tr>
</tbody>
</table>

Agg. = aggregate

The specimens from the first batch were demoulded 24 hours after casting, while 48 hours were allowed for the remaining specimens. All the specimens were cured in a water tank until before testing. At 28 days Ø150 × 380 mm specimens were roughened with an electrical chisel. For each batch of concrete, the compressive strength tests were conducted at 14 days and 28 days (except Batch III), using the Avery 180-ton compression testing machine. The test was in accordance with AS 1012.9 (1999). The pressure was applied at a constant rate of 0.33 MPa/s. Refer to Table 4.2 for the results.

Table 4.2 Concrete compressive strength

<table>
<thead>
<tr>
<th>Batch No.</th>
<th>Age (Days)</th>
<th>Average Strength f',c (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>14</td>
<td>44.0</td>
</tr>
<tr>
<td>I</td>
<td>28</td>
<td>49.6</td>
</tr>
<tr>
<td>II</td>
<td>14</td>
<td>41.5</td>
</tr>
<tr>
<td>II</td>
<td>28</td>
<td>49.6</td>
</tr>
<tr>
<td>III</td>
<td>28</td>
<td>26.6</td>
</tr>
</tbody>
</table>

As shown in Table 4.2, Batch I and Batch II have close results while Batch III had the lowest strength. This was due to the excessive w/c ratio.
4.2.2 Reinforcement Properties and Installation

Galvanised welded steel wire mesh with an aperture size of 12.7 mm x 12.7 mm and a nominal diameter of 1.2 mm was used. The material, distributed by a local firm (Whites Wires), was recommended to have high performance in strength and durability. The detailed investigation of wire mesh will be presented in Chapter 6.

R6 (6 mm diameter plain steel), with a specified strength of 450 MPa was adopted as steel reinforcement. The steel hoops were formed by a local steel fabrication company. To investigate the actual strength and the stress-strain behaviour of lateral steel reinforcement, the tensile strength tests were conducted. Three specimens with an average length of 449 mm and diameter of 6.29 mm were tested using Instron 8033 under the monotonic tensile load until failure. The testing was complied with the Australian Standard AS 1391(2007). The average yield strength was 450 MPa, determined based on 0.2% permanent strain. The stress and strain curves are illustrated in Fig. 4.1.

![Stress and strain curves of R6 steel](image)

**Table 4.3 Properties of R6 steel**

<table>
<thead>
<tr>
<th>Steel Diameter (mm)</th>
<th>Yield Stress (MPa) Average</th>
<th>Ultimate Stress (MPa) Average</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.29</td>
<td>450</td>
<td>524</td>
<td>15.5</td>
</tr>
</tbody>
</table>
Two of the Ø150 × 380 mm specimens were confined with two R6 steel hoops spaced at 140 mm and wrapped with either two or three layers of wire mesh, marked 2L380 and 3L380, respectively; the remaining one was confined with mortar only, marked TMC. No reinforcement was applied. The configuration of Specimens 2L380 and 3L380 are shown in Fig. 4.2. In addition, one Ø150 × 300 mm cylinder from a previous experiment was used for the trial test. This specimen was roughened and reinforced with three the same steel hoops spaced at 110 mm. The distance between the steel ring and the top and bottom of the specimen is 40 mm.

The lateral reinforcement volume ratio of wire mesh and steel hoop was calculated based on AS3600 (2009). It is noted that for wire mesh reinforcement, only the wires in the transverse orientation was considered. The summation of the two ratios was marked $\rho_{s,M}$. In general, the reinforcement ratios were low, ranged from 0.94% to 1.13%, as shown in Table 4.4.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Spacing (mm)</th>
<th>$\rho_s$ (%)</th>
<th>$\rho_{2M}$ (%)</th>
<th>$\rho_{3M}$ (%)</th>
<th>$\rho_{s,M}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2M300</td>
<td>100</td>
<td>0.79</td>
<td>0.38</td>
<td>-</td>
<td>1.18</td>
</tr>
<tr>
<td>2M380</td>
<td>140</td>
<td>0.57</td>
<td>0.38</td>
<td>-</td>
<td>0.94</td>
</tr>
<tr>
<td>3M380</td>
<td>140</td>
<td>0.57</td>
<td>-</td>
<td>0.57</td>
<td>1.13</td>
</tr>
</tbody>
</table>

$\rho_{s,M} = \text{summation of steel hoop and wire mesh reinforcement volumetric ratio}$

The reinforcement installation consists of two steps. Firstly, the steel hoops were placed at the marked location and welded by point-welding. Secondly, wires in the
transverse orientation were applied as confinement. Wire mesh with correct dimension was cut from the roll and installed in a continuous way. A procedure was followed to ensure the core was firmly wrapped with wire mesh: 1) a table was set up; 2) marking the wire mesh pieces based on the circumference of steel reinforced concrete core and straightening the mesh, as shown in Fig. 4.3; 3) placing the core on the desk and tying the mesh to the steel hoops; 4) slowly rolling the concrete core over the mesh and tying the mesh every half of the circumference, approximately 250 mm; and 5) at the overlap area (approximately 80 mm), it was tied at every two grids throughout the whole length at each side of the overlap. Furthermore, the overlap area was labelled for investigation. Refer to Fig. 4.4 for the finalised reinforcement installation. It is noted that there was some imperfection, which was probably related to incomplete manual compaction.

Fig. 4.3 Working desk

Fig. 4.4 Concrete core with reinforcement
Four 20 mm strain gauges (TML, manufactured by Tokyo Sokki Kenkyujo Co., Ltd.) were placed with equal spacing on the mortar circumference at the specimen midlength. Two 5 mm strain gauges were applied opposite to each other at the middle steel hoop of Specimen 2L300. Refer to Fig. 4.5 for the locations of strain gauges.

![Diagram of mortar mix proportions](image)

Note: SG = strain gauge

**Fig. 4.5 Locations of strain gauges _Specimen 2M300**

### 4.2.3 Mortar Mixes

The same type of cement, sand and superplasticiser used for concrete casting were used for the mortar mixes. A number of mortar trials were conducted. The mortar mix proportion of two trials is given in Table 4.5. The first batch mortar was used for the confined specimens. The flow table tests were conducted in accordance with ASTM C 14307(2007). To investigate the compressive strength of mortar, 40 mm × 40 mm × 160 mm prism samples were prepared at the same time, in accordance to AS/NZS 2350.11(2006). Mortar was manually plastered into the reinforcement tube and formed a protective cover of 2-5 mm thick, approximately.

**Table 4.5 Mortar mix proportion by weight**

<table>
<thead>
<tr>
<th>Batch</th>
<th>Cement</th>
<th>Sand</th>
<th>Water</th>
<th>Fly ash</th>
<th>Superplasticiser (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>2.50</td>
<td>0.35</td>
<td>0.18</td>
<td>2.1</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>2.60</td>
<td>0.34</td>
<td>0.20</td>
<td>1.8</td>
</tr>
</tbody>
</table>

Due to equipment constraints, five prismatic samples were tested by Boral Material Services (BMS), in complying with AS/NZS 2350 11 (2006). The properties of mortar are summarised in Table 4.6. It is noted that the mortar compressive strength
tests were conducted at 91 days. The second batch mortar was used for Specimens TMC, 2M380, 3M380 and 2M300.

<table>
<thead>
<tr>
<th>Mortar Batch</th>
<th>Age (days)</th>
<th>Replicate No.</th>
<th>Density (kg/m³)</th>
<th>Compressive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>91</td>
<td>2</td>
<td>2166</td>
<td>58.4</td>
</tr>
<tr>
<td>2</td>
<td>91</td>
<td>3</td>
<td>2223</td>
<td>65.1</td>
</tr>
</tbody>
</table>

### 4.3 Testing of Trial Specimens

Specimens TMC, 2M300, 2M380 and 3M380 were tested using Denison under concentric loading. The loading was performed under displacement control at a speed of 0.5 mm/minute and then adjusted to 1.0 mm/min at a later stage beyond the ultimate load. Before testing each specimen was capped with high strength plaster at each end. For the reinforced specimens, CFRP strips were used for strengthening the ends. Two LVDTs were installed to collect the data for the vertical displacement over the full specimen height. All the strain gauges and LVDTs were connected with a computer and data were collected every three seconds.

#### 4.3.1 Mortar Confined Specimen TMC

A typical brittle failure was exhibited. Tiny vertical cracks initialised at the mid height. The cracks developed quickly throughout the entire height and widened considerably, resulting in the mortar shell breaking into several large pieces. This specimen bulged and shortly the broken shells completely fell off, showing the crushed concrete core. Refer to Fig. 4.6 for the details. This indicated that the bonding condition between the concrete core and the mortar shell was poor, which obstructs the monolithic behaviour of strengthened members. It is noted that this specimen was repaired with HSP near both ends before testing due to imperfection on the surface.
4.3.2 Reinforced Specimens

Specimen 2L300 had a protective cover of 2 mm, approximately. Small vertical cracks initially appeared at the specimen midlength. With the continuous loading, cracks increased. When approaching the ultimate load, thin pieces of mortar crumbled and peeled at the middle height. Large vertical cracks appeared on the top and bottom part of the specimen. Single wires started breaking with a low snapping sound. At the post ultimate load stage, deformation was severe and steel hoop fractured, as shown in Fig. 4.7. The mortar and mesh shell spalled and the concrete core crushed. Reinforcement failure was presented as: 1) lateral wires ruptured in tension; 2) vertical wires buckled; and 3) steel hoop fractured at the welded point.
For Specimens 2L380 and 3L380, the protective cover was approximately 5 mm. The failure mode of the two specimens was similar to that of Specimen 2L300, as shown in Fig. 4.8 and Fig. 4.9. Although large vertical cracking was the dominated phenomenon, scale cracking also appeared on the circumference of Specimen 3L380. Mesh mortar shell ruptured in tension. At the late post ultimate load stage, most of mortar cover peeled. Concrete core crushed and steel hoops fractured. This is because less thick cover facilitates wires to be evenly distributed in the matrix and increase the specific surface of reinforcement ($S_r$), accordingly improving energy absorption.

![Fig. 4.8 Testing of Specimen 2L380](image1)

![Fig. 4.9 Testing of Specimen 3L380](image2)
The axial load-axial displacement of Specimens TMC, 2M380 and 3M380 are shown in Fig. 4.10. This figure shows that for Specimen TMC, shortly beyond the ultimate load, the load dropped approximately 65%. For each reinforced specimen, the ultimate load had been increased considerably, compared to Specimen TMC. Compared to Specimen 2M380, although the ultimate load of Specimen 3M380 did not improve significantly, the descending curve of Specimen 3M380 became more gradually. This indicates the improvement in ductility and the energy absorption capacity. By contrast, Specimen TMC represented a very sharp descending curve. In
general, Specimens 2M380 and 3M380 showed a superior behaviour performance to Specimen TMC in terms of strength and ductility.

The axial load -lateral strain of steel hoop and the circumference of Specimen 2L300 are presented in Fig. 4.11. This figure indicates that at the ultimate load, steel reinforcement was not yield, which was probably affected by the low steel hoop ratio (0.79%) and the expansion of HSC core. Compared to NSC, The expansion of HSC is smaller due to the relatively higher modulus of elasticity and less internal microcracks in HSC (Hong et al. 2006).

![Fig. 4.11 Axial load- lateral strain of Specimen 2L300](image)

### 4.4 Test Results

The details about the failure modes, ductility and the area under the axial load and axial displacement curve (AUC) of Specimens TMC, 2M380 and 3M380 are presented in Table 4.7-Table 4.9. It is noted that the ductility ratios are as defined previously in Chapter 3.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Location</th>
<th>Core</th>
<th>Reinforcement</th>
<th>Cover</th>
</tr>
</thead>
<tbody>
<tr>
<td>TMC</td>
<td>Crushed</td>
<td>-</td>
<td>-</td>
<td>Bulged, completely peeled off in large pieces</td>
</tr>
<tr>
<td>2M380</td>
<td>Crushed held by reinforcement cage</td>
<td>Hoop fractured at welded point</td>
<td>Lateral wire ruptured</td>
<td>Bulged, peeled gradually, most of cover peeled off</td>
</tr>
<tr>
<td>3M380</td>
<td></td>
<td></td>
<td>Vertical wires buckled</td>
<td></td>
</tr>
</tbody>
</table>
Table 4.8 Results of trial tests

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Diameter (mm)</th>
<th>Height H (mm)</th>
<th>Cover t (mm)</th>
<th>Ultimate load $P_u$ (kN)</th>
<th>Load Increment (%)</th>
<th>Ductility Ratio $\Delta u/\Delta 1$</th>
<th>$\mu_y$</th>
<th>$\mu_{.85}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>TMC</td>
<td>170</td>
<td>380</td>
<td>10</td>
<td>678</td>
<td>1</td>
<td>1.08</td>
<td>1.22</td>
<td></td>
</tr>
<tr>
<td>2M380</td>
<td>183</td>
<td>380</td>
<td>5</td>
<td>1118</td>
<td>49.9</td>
<td>1.28</td>
<td>1.73</td>
<td></td>
</tr>
<tr>
<td>3M380</td>
<td>186</td>
<td>380</td>
<td>5</td>
<td>1159</td>
<td>55.5</td>
<td>1.17</td>
<td>1.91</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.9 Area under the axial load-axial displacement curve

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Area under the axial load-axial displacement curve (kN mm)</th>
<th>Finish load $P$ (kN)</th>
<th>Proportion of $P_u$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TMC</td>
<td>788</td>
<td>34</td>
<td>4.9</td>
</tr>
<tr>
<td>2M380</td>
<td>6153</td>
<td>140</td>
<td>12.5</td>
</tr>
<tr>
<td>3M380</td>
<td>9031</td>
<td>191</td>
<td>16.5</td>
</tr>
</tbody>
</table>

As shown in Table 4.8, Specimens 2M380 and 3M380 confined with the same amount of steel hoops had close result in the ultimate load ($P_u$) and the first steel fracture occurred at 58.2% and 56.0% of $P_u$, respectively. For Specimen 3M380, the axial displacement at the first hoop fracture increased significantly than that of Specimen 2M380, indicating the significant increase in deformation. Table 4.9 indicates that wire mesh enhances energy absorption significantly as the AUC value of Specimen 3M380 was 1.47 times that of Specimen 2M380. The two specimens had the identical steel reinforcement ratio. Compared to Specimen TMC, wire mesh combined with steel hoops can enhance the strength and significantly improve the ductility. The increase in the transverse wire volumetric ratio from 0.38% to 0.57% ratio pronouncedly affects the post peak behaviour of confined specimens.

The concrete compressive strength of Specimen 2M300 was unknown. However, it can be reasonably considered close to that of Specimens 2L5C1 and 2L5C2, based on the test results as shown in Table 4.10 and Fig. 4.10 (b).
Table 4.10 HSC Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Hoop Spacing (mm)</th>
<th>Finish Diameter (mm)</th>
<th>Cover t (mm)</th>
<th>Mortar $f_m$ (MPa)</th>
<th>Ultimate load $P_u$ (kN)</th>
<th>Ductility Ratio $\Delta u/\Delta 1$</th>
<th>$\mu_y$</th>
<th>$\mu_{85}/\Delta 1$ at 50% $P_u$</th>
<th>AUC (kN mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2M300</td>
<td>110</td>
<td>177</td>
<td>2</td>
<td>65.1</td>
<td>1680</td>
<td>1.23</td>
<td>1.80</td>
<td>4876</td>
<td></td>
</tr>
<tr>
<td>2L5C1</td>
<td>-</td>
<td>170</td>
<td>5</td>
<td>70.6</td>
<td>1737</td>
<td>1.18</td>
<td>1.22</td>
<td>1997</td>
<td></td>
</tr>
<tr>
<td>2L5C2</td>
<td>-</td>
<td>170</td>
<td>5</td>
<td>70.6</td>
<td>1713</td>
<td>1.22</td>
<td>1.28</td>
<td>2035</td>
<td></td>
</tr>
</tbody>
</table>

It is interesting to compare the post peak behavior among the three specimens. The three specimens had similar geometrical properties; the main difference is that no steel hoop was applied to Specimens 2L5C1 and 2L5C2. In addition, the mortar strength of Specimen 2M300 was lower. Although the three specimens had close ultimate load, Specimen 2M300 outperformed its counterparts in terms of ductility and energy absorption. The steel confinement did not reach the yield strength at the ultimate load, which was mainly attributed to the concrete and mortar. However, steel confinement with a volume ratio $\rho_s = 0.72\%$ considerably affects the post peak behavior. The results indicate that wire mesh is a kind of secondary confinement, compared to steel hoops.

### 4.5 Conclusions

This chapter has provided the investigation on the trial of forming wire mesh, steel hoop and high strength mortar composite. The results suggest that wire mesh combined with steel reinforcement can enhance the strength and significantly improve the ductility. On the other hand, the bonding condition has important influence on the structural behaviour of strengthened members. Increasing one layer of wire mesh (the transverse wire volumetric ratio increased from 0.38\% to 0.57\%), had limited effect on the ultimate load, but significant influence on the post peak behaviour and energy absorption. This investigation has provided basic information for the experiment program and estimation. The following chapter provides the detailed experimental program.
CHAPTER 5 EXPERIMENTAL PROGRAM OF EXPERIMENT III

5.1 Introduction

To have a comprehensive understanding on the behavior of wire mesh and high strength mortar confined normal strength concrete, a comparative experimental program was conducted in the High Bay Laboratory at the University of Wollongong, Australia, unless otherwise specified. This experimental study focused on the effect of the constitutive materials as well as the composite confinement. The program was limited to short cylindrical specimens without longitudinal steel reinforcement. This chapter presents a description of the experimental design for thirty eight specimens tested subjected to concentric short-term monotonic load. The experimental program mainly covers the specimen geometries, constituent materials, specimen fabrication and experiment instrumentation. The details of this experimental program are provided in the following sections.

5.2 Specimen Design

5.2.1 Specimen Categories

It is common to start with the axial compressive behaviour of small-scale plain concrete cylinders when investigating a new strengthening technique for RC columns. As indicated in ACI 440. 2R (2008), the equations for the axial compressive strength of fibre-reinforced polymer (FRP)-strengthened normal strength concrete is considered directly applicable for the situation when the load eccentricity present in the RC member is not greater than 10% of its cross-section height; in the case that the eccentricity is larger, the equations proposed for pure axial compression can also be used with modifications.

Due to the limitation of wire mesh confinement, steel rings were adopted in the third series of experiments. Accordingly, the mortar shell thickness was increased and the specimen diameter was changed to 190 mm from 150 mm. In practice, a frictional force exists between the machine platens and the specimen. It is difficult to eliminate this force (Kim et al. 1999). To avoid the influence of this frictional force on the
compressive strength, also known as the end effect, the length to diameter ratio is required to be not less than 2. Therefore, the height of the specimens of the third series of experiments was adjusted to 380 mm from 300 mm. It should be noted that even with this change, the size difference between the specimens in the third series of experiments and the other two series of experiments is not large.

With ongoing aging of and damage to concrete structures, concrete repair and strengthening has become an urgent challenge and requires much research efforts. Old concrete structures were typically constructed with normal strength concrete (NSC), while high strength concrete (HSC) has been used since 1950s (Shah and Ahmad 1985). To extend the application of this repair technique, NSC was adopted in the third experiment. The three series of experiments in the present study show that wire mesh can be used for strengthening concrete with different strengths.

A total of 40 Ø150 × 380 mm normal strength concrete (NSC) specimens were cast first, among which: six were controls that served as references; 32 were strengthened with different materials; and two were spare specimens. As the second step, the 32 specimens were strengthened with the following materials or composites: 1) modified high strength mortar (MHSM); 2) wire mesh and MHSM; or 3) steel hoops, wire mesh and MHSM. Refer to Table 5.1 for the details of specimen configuration. All the specimens had an identical height of 380 mm and were tested under concentric loading.

The confined specimens were divided into three groups, according to the number of mesh layers installed. The first group was confined with one layer of wire mesh; the second with two layers of wire mesh; and the third with three layers of wire mesh. In each group, there were two specimens strengthened with MHSM only and two specimens confined with both wire mesh and MHSM. Each group consisted of six categories: plain concrete controls; MHSM confined specimens; MHSM and wire mesh strengthened specimens; and specimens strengthened with MHSM, wire mesh and steel hoops at three different spacing. There was an additional category, specimens strengthened with MHSM and steel hoops spaced at 50 mm, for the third group. For all the three groups, each category had two replicates. It is also common
to test only two nominally identical specimens in the existing studies on confined concrete specimens with similar dimensions. Examples of existing studies that adopted this practice include Teng et al. (2007), Jiang and Teng (2007), Wang et al. (2015) and Hadi et al. (2015).

As shown in Table 5.1, the specimens are indicated with alphanumeric labels, in which M and S stand for wire mesh and steel hoop, respectively. The Arabic number in front of the capital letters M and S indicates the number of mesh layers and the spacing of steel hoops, respectively. The last Arabic number of each code shows the number of replicates. The plain concrete controls were labelled as FTC; the specimens strengthened with MHSM only were labelled as MC. The replicates of the concrete controls and MHSM confined specimens are indicated in a consecutive manner.

5.2.2 Specimen Geometry

Thirty two of cast NSC specimens were confined with a 20 mm thick shell comprising MHSM and various amounts of reinforcement. The strengthened specimens had a diameter of 190 mm and remained the same height, 380 mm. This study was focused on the behavior of the axial load and the axial deformation. To examine the effect of lateral confinement, no longitudinal steel bars were applied in this study. The specimen categories are illustrated in Fig. 5.1 and Fig. 5.2.
Table 5.1 Specimen configuration

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Diameter Core, $D_c$ (mm)</th>
<th>Diameter* $D$ (mm)</th>
<th>Cover § $t$ (mm)</th>
<th>No. of Mesh Layer</th>
<th>Hoop Spacing (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FTC1</td>
<td>-</td>
<td>150</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>FTC2</td>
<td>-</td>
<td>150</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>MC1</td>
<td>150</td>
<td>190</td>
<td>20</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>MC2</td>
<td>150</td>
<td>190</td>
<td>20</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1M0S1</td>
<td>150</td>
<td>190</td>
<td>18</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>1M0S2</td>
<td>150</td>
<td>190</td>
<td>17</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>1M100S1</td>
<td>150</td>
<td>190</td>
<td>12</td>
<td>1</td>
<td>100</td>
</tr>
<tr>
<td>1M100S2</td>
<td>150</td>
<td>190</td>
<td>11</td>
<td>1</td>
<td>100</td>
</tr>
<tr>
<td>1M75S1</td>
<td>150</td>
<td>190</td>
<td>11</td>
<td>1</td>
<td>75</td>
</tr>
<tr>
<td>1M75S2</td>
<td>150</td>
<td>190</td>
<td>11</td>
<td>1</td>
<td>75</td>
</tr>
<tr>
<td>1M50S1</td>
<td>150</td>
<td>190</td>
<td>11</td>
<td>1</td>
<td>50</td>
</tr>
<tr>
<td>1M50S2</td>
<td>150</td>
<td>190</td>
<td>11</td>
<td>1</td>
<td>50</td>
</tr>
<tr>
<td>FTC3</td>
<td>-</td>
<td>150</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>FTC4</td>
<td>-</td>
<td>150</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<tr>
<td>MC3</td>
<td>150</td>
<td>190</td>
<td>20</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>MC4</td>
<td>150</td>
<td>190</td>
<td>20</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2M0S1</td>
<td>150</td>
<td>190</td>
<td>16</td>
<td>2</td>
<td>-</td>
</tr>
<tr>
<td>2M0S2</td>
<td>150</td>
<td>190</td>
<td>15</td>
<td>2</td>
<td>-</td>
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<tr>
<td>2M100S1</td>
<td>150</td>
<td>190</td>
<td>9</td>
<td>2</td>
<td>100</td>
</tr>
<tr>
<td>2M100S2</td>
<td>150</td>
<td>190</td>
<td>9</td>
<td>2</td>
<td>100</td>
</tr>
<tr>
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<td>190</td>
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<td>2</td>
<td>75</td>
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<td>150</td>
<td>190</td>
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<td>2</td>
<td>75</td>
</tr>
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<td>2M50S1</td>
<td>150</td>
<td>190</td>
<td>8</td>
<td>2</td>
<td>50</td>
</tr>
<tr>
<td>2M50S2</td>
<td>150</td>
<td>190</td>
<td>9</td>
<td>2</td>
<td>50</td>
</tr>
<tr>
<td>FTC5</td>
<td>-</td>
<td>150</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>FTC6</td>
<td>-</td>
<td>150</td>
<td>-</td>
<td>-</td>
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<tr>
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<td>20</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
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<td>190</td>
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<td>50</td>
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<tr>
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<td>190</td>
<td>5</td>
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<td>50</td>
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<td>0M50S2</td>
<td>190</td>
<td>150</td>
<td>14</td>
<td>-</td>
<td>50</td>
</tr>
</tbody>
</table>

Note: *outermost diameter of specimens; § mortar protective cover
Fig. 5.1 Specimen categories_1 (dimensions in mm)
Fig. 5.2 Specimen categories_2 (dimensions in mm)
5.2.3 Steel Reinforcement Spacing and Volumetric Ratio

Lateral reinforcement can improve both the strength and the ductility of the interior concrete (Warner et al. 1999). This experimental program is focused on the effectiveness of lateral confinement to NSC core. The main variables of the reinforcement are the steel reinforcement spacing and the number of wire mesh, which are quantified by the volumetric ratio.

5.2.3.1 Steel Reinforcement Spacing

Due to the dimension of concrete core, plain steel with a diameter of 6 mm (R6) was adopted for forming steel hoops. R6 was the minimal diameter requirement for fitment and helix according to Clause 10.7.4.3 (AS3600 -2009). However, this requirement is based on the longitudinal bar diameter. Previous research (Sheikh and Toklu 1993) suggested that steel hoop confined concrete outperforms spiral confined concrete as steel hoop behaves individually and the fracture of steel hoop might not affect the confinement provided by the remaining hoops.

According to Clause 10.7.4. (AS3600 -2009), the spacing of fitments is related to the diameter of longitudinal steel bars. Considering the single bars case, the spacing of steel hoops should not greater than the smaller of the column diameter or 15 times of the diameter of single bars. No longitudinal steel bars were applied in this study. Assuming applying relatively small diameter bars like 8 mm, the maximum space would be 120 mm. According to Cl. 10.7.3 (AS3600-2009), for columns where \( f'c \) is greater than 50 MPa, a minimum effective confining pressure to the core is set to be \( 0.01f'c \). This requirement is deemed to be satisfied for circular sections if the spacing of transverse reinforcement not greater than Eq. 5.1.

\[
s_{\text{max}} = \frac{100A_{b,\text{fit}}f_{yw,f}}{d_s f'_c} \tag{5.1}
\]

where \( A_{b,\text{fit}} \) is the cross-sectional area of one leg of the transverse reinforcement; \( f_{yw,f} \) is the yield stress of the transverse reinforcement; \( d_s \) is the overall dimension measured between centre-lines of the outermost reinforcement, \( f'_c \) is the average compressive (cylinder) strength of concrete at 28 days. As the target concrete
compressive strength was 32 MPa, assuming $f'_{c} = 50$ MPa, $f_{sy,f} = 500$ MPa, the maximum space would be 163 mm.

On the other hand, considering the seismic requirements of Cl. 21.6.4 (ACI 318M-08), a maximum space of 47.5 mm is required for the steel hoops along the length of the specimens. A range of spacing, 100 mm, 75 mm and 50 mm was adopted for this study. The reinforcement details of the confined specimens are shown in Fig. 5.3.

![Fig. 5.3 Steel reinforcement spacing (dimensions in mm)](image)

5.2.3.2 Steel Reinforcement Volumetric Ratio

The volumetric ratio of transverse steel reinforcement (steel fitments) relative to the volume of the core ($\rho_s$), is an important parameter for quantifying lateral steel confinement and determined using Eq. 5.2 (AS3600-2009):

$$\rho_s = \frac{A_{b,fit} \times \text{total perimeter of fitments crossing the section}}{A_c \times s}$$  (5.2)

where $A_c$ is the cross-sectional area bounded by the centre-line of the outermost fitments. The volumetric ratio for the spacing of 100 mm, 75 mm and 50 mm, respectively, was 0.72%, 0.97% and 1.45%, respectively.

5.2.4 Wire Mesh Reinforcement Ratio

It is reasonable to consider lateral wires as closely spaced circular steel hoops with small diameter. Wires in the lateral direction were quantified by the same approach. The perimeter of lateral wires was calculated based on the overall dimension of the
The volumetric ratio of wire mesh, which takes into account only lateral wires, was calculated using Eq. 5.2. For the specimens confined with one, two and three layers of wire mesh without steel reinforcement, the volumetric ratio of wire mesh was 0.21%, 0.41 and 0.59%, respectively. For the specimens confined with one, two and three layers of wire mesh and with steel reinforcement, the volumetric ratio of wire mesh was 0.19%, 0.37% and 0.53%.

5.2.5 Combined Reinforcement Volumetric Ratio

Table 5.2. Reinforcement Ratios

<table>
<thead>
<tr>
<th>Reinforcement Type</th>
<th>Steel Hoop Spacing (mm)</th>
<th>Steel Volumetric Ratio $\rho_1$ (%)</th>
<th>Wire mesh Volumetric ratio $\rho_{1M}$ (%)</th>
<th>Wire mesh Volumetric ratio $\rho_{2M}$ (%)</th>
<th>Wire mesh Volumetric ratio $\rho_{3M}$ (%)</th>
<th>Wire mesh and hoop Volumetric ratio $\rho_{s+1M}$ (%)</th>
<th>Wire mesh and hoop Volumetric ratio $\rho_{s+2M}$ (%)</th>
<th>Wire mesh and hoop Volumetric ratio $\rho_{s+3M}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wire mesh</td>
<td>0</td>
<td>-</td>
<td>0.21</td>
<td>0.41</td>
<td>0.59</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<td>Wire mesh Steel hoop</td>
<td>100</td>
<td>0.72</td>
<td>0.19</td>
<td>0.37</td>
<td>0.53</td>
<td>0.91</td>
<td>1.09</td>
<td>1.25</td>
</tr>
<tr>
<td></td>
<td>75</td>
<td>0.97</td>
<td>0.19</td>
<td>0.37</td>
<td>0.53</td>
<td>1.16</td>
<td>1.34</td>
<td>1.50</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>1.45</td>
<td>0.19</td>
<td>0.37</td>
<td>0.53</td>
<td>1.64</td>
<td>1.82</td>
<td>1.98</td>
</tr>
<tr>
<td>Steel hoop</td>
<td>50</td>
<td>1.45</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 5.2 indicates that the total reinforcement ratio improved significantly with three layers of wire mesh.
5.3 Constituent Materials

5.3.1 Normal Strength Concrete (NSC)
HSC and MSC were adopted for the first experiment and second experiment, respectively. A local ready-mix NSC was used for the third experiment, with a maximum aggregate size of 10 mm.

5.3.2 Modified High Strength Mortar
Mortar was mixed in the laboratory. Portland cement (ASTM Type I), Natural sand with a fineness modulus value of 2.6 (AS1289.3.6.1-2009) and POZZOLITH 370 PC (superplasticiser) were used in the mixture. In addition, a SBR acrylic based bonding and polymer modifying agent, EMACO 157, was also used. EMACO 157 improved consistency and workability of the matrices and facilitated mortar passing through small apertures and sticking to the concrete core. Apparently, EMACO 157 enhanced the bonding between the core and the shell as well as the bonding of the constituent materials. In a composite section, the bonding between the constituent materials enables the stress transfer in the constituent materials (Lodi et al. 2010). Recall that due to the increase in the mortar shell thickness, the strengthening shell of Specimen TMC (with no bonding agent applied) spalled completely just after the ultimate load, as shown in Chapter 4. To avoid this very brittle failure, EMACO 157 was used in the third series of experiments. Flow table test and the compressive strength test were conducted for the mortar mixes. Refer to Section 6.6 in Chapter 6 for details.

The mortar strength for all the specimens was originally designed as identical. However, due to the limitation of equipment capacity, two batches of mortar were mixed for each group. In total, six batches of mortar were cast, marked as FMB1, FMB2, FMB3, FMB4, FMB5 and FMB6. For example, FMB1 and FMB2 were applied to the first group specimens. It is noted that a series of mortar trial mixes had been conducted. The results of the trial and the final mixes will be provided in the next chapter.
5.3.3 Steel Reinforcement

The steel hoops were made out of plain steel with a nominal diameter of 6 mm. The steel reinforcement was specified with yield strength of 450 MPa. The steel reinforcement was outsourced from a local steel fabrication company where the steel reinforcement formed into steel hoops with an inner diameter of 150 mm. The steel hoops with open ends were welded in the laboratory by a qualified technician. The tensile strength investigation was conducted on R6 steel bars, spot-welded R6 steel bars and stitch-welded R6 steel bars. Refer to Chapter 6 for details.

5.3.4 Welded Steel Wire Mesh

Welded steel wire mesh with an aperture size of 12.7 mm × 12.7 mm (E12.7WM) was adopted for this study. The welded steel mesh had been hot dip galvanised after manufacture. The wire mesh was recommended to have a high strength and durability. The wire mesh was packed in a 1200 mm high and 30 m long roll, distributed by a local firm (Whites Wires). According to the information provided by the supplier, the diameter of single wire was 1.24 mm. However, the measured average diameter of lateral wires was 1.14 mm. The measured density of the wire mesh, ρw, was 5152 kg/m³.

To investigate the tensile strength of wire mesh, the tests were carried on single-wire samples in both transverse and longitudinal directions. In addition, six mesh coupons were prepared and tested. Wire mesh coupons were obtained from the same roll and prepared in accordance with ACI 549.1R (1993). Each of the mesh coupons had a total length and width of 400 mm and 80 mm, respectively; the end had a dimension of 80 mm x 80 mm, as shown in Fig. 5.4. Each end of the coupons was embedded in mortar with an additional layer of wire mesh (ACI 549.1R-1997). The thickness of mortar ends was 20 mm.
Ply timber mould with a thickness of 18 mm was made for embedding the ends into mortar, as shown in Fig. 5.5. To prevent leakage, the edges inside of the timber mould were sealed with silicon, 48 hours before embedding the coupon. Before casting, Vaseline was evenly applied to the inside of each mould. Vibration was applied to achieve a good compaction. After casting specimens were kept in mould for 48 hours covered with plastic sheet. After moulding each end of the coupons was enclosed with cling wrap and cured at room temperature before test. During the curing period, specimens were watered regularly for 28 days. Fig. 5.6 shows the mesh coupons embedded in mortar.
5.3.5 Mesh-Encapsulated Mortar Specimens

To determine the strength of mesh-encapsulated mortar (ferrocement plate) under tensile load, three ferrocement plates were prepared according to the procedure suggested by ACI Committee 549 (ACI 549.1R-88) for each batch of mortar (FMB1-MB6) with the corresponding number of wire mesh layers. The dimension of ferrocement plate was 400 mm long and 80 mm wide. The thickness was 20 mm, same as the mortar and reinforcement shell for the confined specimens. Three similar plywood mould with a dimension of 606 mm × 436 mm were used for casting the ferrocement plates. Refer to Fig. 5.7 for details. The moulds were made in a similar way as for mesh coupon samples.
Mesh coupons were cut into right size from the same roll of wire mesh in the transverse direction. Each end was reinforced with an additional 80 mm x 80 mm wire mesh, tied firmly using coiled steel wires with similar diameter as steel wires. Mortar was cast in two layers. A thin layer of 10 mm thick mortar was evenly placed into each component of the mould. After the mesh coupon was placed on the top of mortar and slightly pressed down, the second layer of mortar was placed. Sufficient vibration was conducted to facilitate mortar compacting. To ensure a smooth and even finish, a trowel was used. The cast plates are shown in Fig. 5.8. Covered with a plastic sheet, the specimens were kept in mould for 48 hours. After being removed from the mould, the ferrocement plates were enclosed with cling wrap and were watered regularly for 28 days.
5.4 Fabrication of Confined Specimens

The following sections provide the details on the specimen preparation, covering the formwork for concrete and mortar casting, concrete and mortar casting and curing, forming and placing reinforcement and strain gauge installation.

5.4.1 Formwork for Concrete Casting

The moulds for concrete core were made of PVC pipe (SN4), having an inner diameter of 150 mm and a thickness of 5 mm. In accordance with the required length, the PVC pipe was cut into 380 mm long pieces with an electric saw. For each mould, the edge of both ends was smoothened manually. To ensure cast quality, a plywood frame was designed to facilitate erecting and stabilising the PVC moulds. The frame mainly consisted of a sturdy timber and plywood base, two 18 mm plywood plates with an identical dimension of 1320 mm x 1135 mm. The two plates were formed the top and the bottom part of the frame. Each of the top and bottom plywood piece was cut identically with 7 x 6 circular openings and each opening had a diameter of 160 mm. The bottom plywood plate was screwed on the base. The top plywood plate was erected by plywood struts (150 mm × 375 mm × 18 mm) around the four corners of the base plate. To enhance the stiffness, an additional timber strut was placed at the middle along each side. Refer to Fig. 5.9 for details.

Fig. 5.8 Cast mesh-encapsulated mortar specimens (dimensions in mm)
PVC moulds were placed into the frame one by one. The bottom of each PVC mould was sealed with silicon to prevent concrete leakage. To keep it clean, the frame was covered with a plastic sheet after cleaning. Before concrete casting the inside of each mould was evenly greased with Vaseline. To ensure moulds would be removed easily after casting, all the screws were also covered with Vaseline.

5.4.2 Concrete Casting and Curing

The formwork for concrete casting was transported to the Strong Floor Laboratory for casting. The slump test was conducted when the concrete arrived. The result of slump test was 175 mm, complying with AS1012.3.1 (1998). In addition to the 40 cylindrical specimens, twenty Ø100 × 200 mm cylinders and six Ø150 × 300 mm cylinders were cast. The two types of cylinders were tested for the compressive strength at 7 days, 28 days, and 56 days, respectively.

Wheel barrows and scoops were used for transferring the concrete in the laboratory and placing concrete into the moulds, respectively. The cast was in accordance with AS1012.8.1 (2000). Concrete was compacted with a 20 mm immersion vibrator, which was immersed vertically into concrete for 5-15 seconds (no bubbles came out). A trowel was used for smoothening finish. Refer to Section 3.2 for concrete curing. Fig. 5.10 and Fig. 5.11 show concrete cast and curing, respectively.
5.3 Forming and Installing Steel and Wire Mesh Reinforcement

Before installing steel reinforcement, each core was roughened with an electrical drill evenly around the circumference, until coarse aggregates clearly appeared, as shown in Fig. 5.12. Two 20 mm strain gauges (TML, manufactured by Tokyo Sokki Kenkyujo Co., Ltd.) were placed opposite each other at the mid height. Before roughening the areas for strain gauges were marked, as shown in. Extra attention was given to these areas during the roughening process.

Note: SG=strain gauge

Fig. 5.12 Roughened concrete core
After roughening, concrete cores were marked according to the corresponding steel hoop spacing. R6 steel hoops with an inner diameter of 150 mm were placed on the marked locations. Concrete cores were horizontally placed on a working table for welding. As no longitudinal steel bars were installed, each hoop was stitch-welded at the end with a 40 mm-long steel piece, which was cut from the spare steel hoops. The welding process was completed by a qualified technician in the laboratory. Refer to Fig. 5.13 for details.

![Fig. 5.13  Marked concrete core and welded steel hoops](image)

When placing the steel hoops, care was needed to avoid placing the welded areas along the same vertical location. After the installation of steel hoops, strain gages were glued onto the cores and lateral steel hoops at the specified areas. Refer to Section 5.5 for the process of strain gauge installation.

Wire mesh was formed into a tube in a continuous manner. A working table was set up first for forming wire mesh tubes, as shown in Chapter 4. Large pieces of wire mesh were cut off from the roll, which were cut into exactly required dimensions on a cutting machine. To facilitate investigating the overlap after test, the edge of each piece was sprayed with yellow paint. A cylinder (Ø150 mm) made out of a number of round plywood plates with the same steel hoops was used to roll the wire mesh to form a tube. Those round plywood pieces were cut off from the larger plywood plates used for erecting PVC moulds.
The process of forming a wire mesh tube was similar to that for the trial specimens as mentioned in Section 4.2.2, Chapter 4. With regard to the strain gauges with protective coating on the core and/or the steel hoop, a wire mesh tube was formed with a slightly larger diameter. To make it easy to adjust the tube for installation, fewer wires were used to tie the mesh. Before placing the wire mesh cage onto the core, each cage was fastened by three galvanised hose clips, as shown in Fig. 5.14(a). Extra care was required for the strain gauges and cables while installing the mesh cages tubes. The wire mesh tube was closely installed on the core by adjusting the hose clips. Refer to Section 4.2.2 for wire mesh overlap. Refer to Fig. 5.14 (b) and (c) for details. The steel wires used as ties were painted in blue to facilitate the after-testing investigation.

![Wire mesh tube installation](image)

**5.4.4 Formwork for Mortar Casting**

To provide a uniform mortar shell, moulds were used for HSM casting. The moulds were made of foam and purchased from a local company. The foam moulds were not used for the specimens confined with steel hoops and three layers of wire mesh due to the very limited space between the outermost wire mesh and the inside of moulds. Refer to the cover thickness in Table 5.1. Therefore, mortar was plastered to Specimens 3M100S1, 3M100S2, 3M75S1, 3M75S2, 3M50S1 and 3M50S2.

The mould was cut from a 240 mm × 240 mm × 380 mm foam prism. A cylinder with a diameter of 190 mm and a height of 380 mm was removed from the central of the foam prism. Refer to Fig. 5.15 for details. A steady working table was set up for
mortar casting. Two rows of foam moulds were set up along the length of table at both sides. Two timber stubs with almost the full length of the table were screwed onto the base at each side. For each row two short timber stubs were placed at the end to form a closed area for erecting the moulds. To get the cables out, a hole was drilled on two sides facing each other. A small piece of plastic pipe was placed at the edge of hole to protect the cable.

After the moulds were placed in two rows, steel rod was fixed to the four corners of each row to support the steel beams, which was made of equal anger steel and screwed on the rods. Two large clips were placed between the two rows to enhance the moulds during casting. Refer to Fig. 5.16 details. Before mortar casting the inside of each mould was evenly greased with Vaseline.

![Fig. 5.15 Foam mould for mortar casting (dimensions in mm)](image)

![Fig. 5.16 Setup for mortar casting](image)
5.4.5 Mortar Casting and Curing

As mentioned earlier in total six batches of mortar cast, marked as FMB1, FMB2, FMB3, FMB4, FMB5 and FMB6. Before casting mortar, all the concrete cores with reinforcement cages were cleaned using high pressure air pump. Just before casting, a thin layer of EMACO 157 was brushed onto the roughened circumference of core. Cores were placed and centralised in the moulds. Mortar was cast in two layers. An immersion vibrator was used to compact mortar. However, due to limit space between wire mesh and the interior circumference of moulds, the vibrator was held on the elevation of foam moulds. After casting, the mortar surface was levelled with a trowel. For the Specimens 3M100S1, 3M100S2, 3M75S1, 3M75S2, 3M50S1 and 3M50S2, mortar was manually plastered. Plastic strips (380 ×20 ×5 mm) were used for controlling the protective cover. A specially curved trowel with a radius of 95 mm, the same as the core, was used for mortar plastering.

A few hours after casting, all the specimens were covered with plastic sheets on top to prevent moisture loss and allow continual hydration of cement. The foam moulds were removed three days after casting. For curing each confined specimen was individually wrapped up with plastic cling and watered regularly for 28 days. An additional layer of plastic sheet was placed on top of the specimens.

5.5 Strain Gauges

5.5.1 Strain Gauge Locations

To investigate the dilation of confined core and the relationship between stress and strain of steel hoops, strain gauges were placed before mortar casting. The strain gauges (TML) were manufactured by Tokyo Sokki Kenkyujo Co., Ltd and supplied by Bestech Australia Pty Ltd. Two 20 mm strain gauges were glued to each concrete core. The two strain gauges were placed opposite each other at the mid height. Two 5 mm strain gauges were placed on the steel hoop opposite to each other at the mid height. For the specimens spaced with steel at 75 mm, the 5 mm strain gauges were 50 mm below the mid height, as shown in Fig. 5.17. The two 5 mm strain gauges were located on the outside of the lateral reinforcement. It is noted that using strain
gauge might not be the best option for investigating the strain on concrete; however, it is the only available solution due to the experiment constraints.

![Diagram of strain gauge locations](image)

**Fig. 5.17 Locations of strain gauges (dimensions are in mm)**

### 5.5.2 Installation of Strain Gauges

In order to install strain gauges, a sufficient length of steel was polished with ‘Emerg’ 600 grit sand paper to provide a smooth surface for transverse strain gauge installation. Before the strain gauge was glued to the reinforcement, the ground and polished area of the steel reinforcement was cleaned with acetone. To make sure the
super strong glue was uniformly distributed, and to avoid skin contact, an application method of using sticky tape was adopted.

For each strain gauge, sticky tape was applied on the top, and then each strain gauge was stuck to the cleaned area of the steel hoop. The next step involved lifting one side of the tape, putting one or two drops of glue under the strain gauge, then pressing the sticky tape evenly over the glued strain gauge for 10 seconds. The sticky tape was then gently peeled away, as shown in Fig. 5.18. As the last step, non-corrosive silicone (Plumbers 780) was applied on top of the now firmly attached strain gauge, including the end cables. Forty eight hours was allowed for silicone to harden.

![Fig. 5.18 Strain gauge (5 mm)](image)

For each concrete core, two 20 mm strain gauges were glued to the circumference at the midheight, following a similar procedure as described in Chapter 3. Refer to Fig. 5.19 for the installation of 20 mm strain gauges. The cable lengths were tied to the steel hoop for a certain distance and then attached to the wire with fishing line. The cables from all the individual strain gauges were braided, so that all the braided cables could be placed into a hose to be taken out from the foam mould. For Specimens MC1-MC6, additional length of cable was glued onto the circumference due to lack of reinforcement to attach the cables.
5.5.3 Strain Gauges on the Circumference

For Specimens FTC1-FTC6, two 100 mm strain gauges were vertically placed opposite each other at the midlength. Four 50 mm strain gauges were placed equally around the circumference at the midlength. For the confined specimens, two sets of strain gauges were glued onto the mortar circumference at the specimen midlength; each consisted of one 100 mm strain gauge in the longitudinal orientation and one 50 mm strain gauge in the transverse orientation.

5.6 Strengthening Ends

High strength plaster called Hydrostone was used for capping each end of the specimens. Due to the height of the specimens, steel reinforcement was uniformly distributed over the full length. Therefore, CFRP strips with a width of 37.5 mm were used to strengthen each end of the confined specimens. Two layers of CFRP were applied with an overlap of one quarter of the specimen diameter. The procedure was the same as described in Chapter 3. The epoxy (manufactured by West System), consisted of West 105 epoxy resin and West 206 slow hardener at a ratio of 5:1.

5.7 Loading and Instrumentation

All the specimens were tested under concentric loading using the Denison machine, which has a maximum specimen height of 1000 mm and an ultimate load carrying capacity of 5000 kN. Two Linear Variable Differential Transducers (LVDTs) were adopted to measure the axial displacement. The two LVDT were attached onto the base plate diagonally and connected to the computer, as shown in Fig. 5.20.
Before the test started, the machine was calibrated and the specimen was adjusted to the centre. The LVDT reading was also adjusted close to zero. The loading was performed continuously under displacement control at a strain rate of 0.5 mm/minute (0.0083 mm/s), which was then adjusted to 1.0 mm/minute (0.0167 mm/s) in the late post peak load region, and until final failure. The end point position of the load cell was set as 20 mm. The LVDTs and all the working strain gauges were connected to the computer. Before connecting each strain gauge was checked. Data was recorded every two seconds. To assist with identifying cracks, cracks were marked with black markers during the test. It is noted that a black circle was marked at the mid height of each specimen for locating lateral strain gauges.

![Fig. 5.20 Compression testing setup](image)

**5.8 Sequence of Forming Final Specimens**

As described above, the procedure for specimen preparation comprises concrete casting, concrete core roughening, steel reinforcement installation, strain gauges installation, wire mesh reinforcement installation, mortar casting, final specimens preparation. The main steps are illustrated in Fig. 5.21.
This chapter has provided the detailed experimental procedure, with emphasis on steel and wire mesh reinforcement formation and installation, concrete and mortar casting. In addition, the preparation of mesh coupons and mesh encapsulated mortar specimens is also described. The following chapter presents the results of the preliminary testing.
CHAPTER 6  PRELIMINARY TESTING OF CONSTITUENT MATERIALS

6.1 Introduction
To determine the properties of the constituent materials of the confined specimens, a series of tests were conducted, including 1) tensile strength tests of steel reinforcement, single wire and mesh coupons; 2) direct tension tests of mesh encapsulated mortar plates; 3) compressive strength test of concrete and mortar, and 4) modulus of elasticity of concrete; 5) an approach to achieve mortar matrices with high strength, flow ability and bonding ability was developed through trial mixing. Discussion of the test results is provided in the following sections.

6.2 Investigation of Single Wires and Mesh Coupons
Welded steel wire mesh with an aperture of 12.7 mm × 12.7 mm (E12.7WM) was adopted as the confinement. The tensile strength is one of the most important properties for lateral confinement. To examine the tensile strength of steel wire mesh, both single wire samples and mesh coupons were prepared in accordance with ACI 549.1R-9. Furthermore, single wires in both the lateral and the vertical directions were investigated.

6.2.1 Single Steel Wires
As wire strands in the longitudinal direction might be different from those in the transverse direction, single wires cut from the same roll of mesh in two directions were investigated. The diameter of single wire samples was measured manually with a digital Vernier calliper at three different locations along the sample, two at the each end and one in the middle. The average value was adopted as the diameter of the wire sample. In this study the average diameter of lateral wires was 1.14 mm.

Following the suggestion (Naaman 2000), the single wire samples were prepared. To prepare the single wire samples, a piece of mesh was cut into single strand one by one at the correct length. The tensile strength tests of single wires were conducted
using Instron 4302 in the Structures Laboratory, as shown in Fig. 6.1(a). The capacity of Instron 4302 is 10 kN at a speed of 0.4 mm/s. The ultimate stress of single wires in the transverse and longitudinal orientation was 620 MPa and 602 MPa, respectively, as shown in Table 6.1. Based on a significant number of tests, it is found that the ultimate stress of single wire was more variable in the longitudinal direction. However, compared with the conventional steel reinforcement, it is reasonable to consider wire mesh reinforcement is uniformly distributed in the composite.

![Image](a) Single wire testing (b) Tested single wires

Fig. 6.1 Tensile strength test of single wires

<table>
<thead>
<tr>
<th>Orientation</th>
<th>Diameter (mm)</th>
<th>Ultimate Stress (MPa)</th>
<th>Average</th>
<th>Standard Deviation</th>
<th>Coefficient Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse</td>
<td>1.14</td>
<td>620.0</td>
<td>19.5</td>
<td></td>
<td>0.031</td>
</tr>
<tr>
<td>Longitudinal</td>
<td>1.15</td>
<td>601.7</td>
<td>85.8</td>
<td></td>
<td>0.143</td>
</tr>
</tbody>
</table>

6.2.2 Welded Steel Wire Mesh Coupons

As mentioned in the previous chapter, six mesh coupons with a dimension of 400 mm × 80 mm were prepared (Naaman 2000). The gauge length was 240 mm. Each end had a dimension of 80 mm × 80 mm and thickness of 20 mm. Recalled that the test results of 5-strand and 7-strand mesh coupons of Experiment II indicated that the
difference in dimension had marginal influence on the results. The tensile strength test of mesh coupons was conducted using Instron 8033 at a speed of 0.3 mm/min up to the ultimate load and then increased to 0.5 mm/min, as shown in Fig. 6.2. The test machine has a capability of 500 kN. During the test it was observed that wire strands at the edge usually ruptured first. The strands ruptured one by one randomly. As a consequent, the load dropped. In most of the cases all the wire strands ruptured at the same height. However, sometimes a single wire could rupture at a different but close position.

![Fig. 6.2 Mesh coupon test](image)

The average test results are summarised in Table 6.2. The ultimate stress (strength) of wire mesh was determined by dividing the ultimate load with the total cross section areas of wires in the vertical direction. The load and elongation curves are shown in Fig. 6.3, while the stress and strain curves are illustrated in Fig. 6.4.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Gauge Length (mm)</th>
<th>Average Ultimate Tensile Load $T$ (kN)</th>
<th>Average Ultimate Stress $f_{uw}$ (MPa)</th>
<th>Average Yield Stress $f_{yw}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mesh coupon</td>
<td>240</td>
<td>4.18</td>
<td>606</td>
<td>562</td>
</tr>
<tr>
<td>Standard deviation</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Coefficient variation</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
As shown in Table 6.2, the ultimate stress and the yield stress of mesh coupons were 606 MPa and 562 MPa, respectively. The strength of mesh coupon is considered more reflecting the mesh system.

![Fig. 6.3 Tensile load and elongation of mesh coupons](image)

Similar to the patterns exhibited by the mesh coupons for Experiment II (refer to Chapter 3 for details), two patterns were presented. As shown in Fig. 6.3, in the majority cases after reaching the ultimate load, wire strands started rupturing one by one. By contrast, a very long load plateau and simultaneous rupture of wire strands was illustrated by Specimen Mesh5. This type of failure also occurred in spirally confined specimens (Pessiki et al. 2001). It is noted that Mesh2 was not included in this figure as one wire strand ruptured earlier before all the wire strands reached the ultimate load. Due to the large number of wires, the early rupture occurred more often for the confined specimens, which reflects the quality of wire mesh reinforcement. In general, the quality of wires is more variable compared to steel reinforcement.
Fig. 6.4 shows that no clear yielding point was displayed in any of the specimens. The arbitrary yield stress determined by the offset method (0.2 %) is 562 MPa while the average ultimate stress (strength) is 606 MPa. Specimen Mesh5 exhibited a large strain while being close to the similar ultimate stress compared with its counterparts. The average arbitrary yield stress of mesh coupons was adopted for determining the confinement strength.

6.3 Mesh Encapsulated Mortar Specimens

6.3.1 Testing of Mesh Encapsulated Mortar Specimens

As mentioned earlier, the reinforcement characteristics significantly influence the confinement effect and the failure mode of confined concrete. The tensile strength of mesh-encapsulated mortar (ferrocement) plates was investigated. During the mortar casting process for the strengthened specimens, three ferrocement plates were simultaneously cast with the corresponding layers of wire mesh from the same roll. The specimens were labelled with alphanumeric characters: the first Arabic number indicates the number of wire mesh layers; letter L and W strand for layer and wire mesh, respectively; letter M stands for mortar; the last Arabic number indicates the number of the replicates; number 1-3 imply the first batch mortar for that group of strengthened specimens while number 4-6 indicate the second batch mortar of that group.
All the ferrecement plates were tested using Instron 8033 under the displacement control, as shown in Fig. 6.5. The machine was calibrated before testing. For Specimens 1LWM1, 1LWM2 and 1LWM3, the tests were carried at a speed of 0.5 mm/min, while the rest were tested at a speed of 0.3 mm/min.

![Image of test setup](image)

(Dimensions in mm)

Fig. 6.5 Mesh encapsulated mortar plate under direct tension

Before testing, all the specimens were marked. Refer to Fig. 6.5 for details. Two sets of axial strain readings were obtained. The first was based on the reading of an extensometer (Made by Epsilon Technology Group), which was mounted onto the plate surface; while the second was calculated by dividing the displacement by the full gauge length. For the first method, in order to place the extensometer two steel hooks were glued (with epoxy adhesive glue, Araldite) onto the specimen surface. The extensometer has a gauge length of 101.6 mm. However, the first method is affected by the location of the cracks. The results suggest that the uniform strain obtained by the second method is more suitable. In addition, one 100 mm strain gauge was also vertically glued on the back of the specimen along the centre line to record the axial strain. Unfortunately this type of results was found unreliable, which confirmed the report by other researchers (Arif et al. 1999).
6.3.2 Behaviour of Mesh Encapsulated Mortar Plates under Direct Tension

During the test it was observed that cracking usually initiated within 50 mm above or below the mid height. However, the initial cracking did not necessarily develop into a well-defined crack, where most of the wire strands ruptured and the width, which was the largest and orthogonal to the loading direction, continued throughout the plate thickness. Usually a couple of cracks appeared and gradually widened.

The cracking in mortar matrix caused the load carrying capacity to drop, which was reflected by the jagged section in the load and elongation curve. Eventually a well-defined crack formed. While mortar cannot contribute to the load carrying capacity, wire mesh sustained the load and reached the ultimate load. The load dropped gradually after passing the ultimate load due to the rupture of wire strands. For the ferrocement plates with one layer of wire mesh, wires ruptured one by one or a few wire strands ruptured simultaneously. It is noted that not all the wire strands ruptured at the well-defined crack. A single wire can rupture at a close location. This phenomenon is similar to wire mesh coupon. For the ferrocement plates with two or three layers of wire mesh, the load dropped faster after passing the ultimate load, especially the specimens with three layers of wire mesh. This indicates wire strands ruptured simultaneously.

The tests show that with the increase of the number of mesh layers, the number of transverse cracks increased, or in other words, the spacing of the cracks decreased. However, the maximum width of cracks tended to reduce. Some selected tested ferrocement plates are shown in Fig. 6.6. The crack pattern might be improved if the mesh layers were even spaced in the matrix instead of being placed in the middle matrix, as the specific surface would have increased. The specific surface of wire reinforcement has the most influence on the crack spacing and width (Shah and Naaman 1978). To distribute wire mesh uniformly in mortar matrix, special spacers might be placed between the mesh layers, which will enhance the specific surface of reinforcement ($S_r$).

However, for the present investigation, wire mesh was formed into tube with close layers as shown in Chapter 5. It is noted that some surface damage was caused by
removing the hooks. The load and elongation plots of the majority of the ferrocement plates are shown in Fig. 6.7-Fig. 6.9. The tested

Fig. 6.6 Tested ferrocement plates

Fig. 6.7 Load and elongation of ferrocement plates with one layer of wire mesh
Under tension the behaviour of ferrocement is controlled by wire mesh reinforcement (Naaman 2000). The volume fraction and specific surface of reinforcement are commonly used to characterise the reinforcement in ferrocement. The volume fraction of reinforcement, $V_f$, is the total volume of reinforcement divided by the volume of composite. For square wire mesh, $V_f$, can be determined by Eq. 6.1. Specific surface of reinforcement, $S_r$, is defined as the total bonded area of reinforcement divided by the volume of composite and can be determined by Eq. 6.2 (ACI549.1R 1993). The specific surface of reinforcement is an important
characteristic of ferrocement, which has considerable influence on the maximum number of cracks and the average cracking spacing and width. The volume fraction for ferrocement plates reinforced with one, two and three layers of wire mesh is 0.0080, 0.0161 and 0.0241, respectively; the specific surface for ferrocement plates reinforced with one, two and three layers of wire mesh is 0.0282, 0.0564 and 0.0846, respectively.

\[ V_f = \frac{n \pi d_w^2}{2 h s_w} \]  \hspace{1cm} (6.1)

\[ S_v = \frac{4 V_f}{d_w} \]  \hspace{1cm} (6.2)

where \( n \) is the number of layers of mesh; \( d_w \) is the diameter of the wire; \( h \) is the thickness of ferrocement section; \( s_w \) is the centre-to-centre spacing of lateral wires.

6.3.3 Test Results of Mesh Encapsulated Mortar Plates

The direct tensile test results of wire mesh samples and ferrocement plates are summerised in Table 6.3. The strength of the ferrocement composite was calculated by dividing the ultimate load with the cross section areas of mesh mortar composite, while the strength of wire mesh was calculated by dividing the ultimate load with the cross sectional area of wire strands in the direction considered (Naaman 2000).

<table>
<thead>
<tr>
<th>Specimen Type</th>
<th>Mesh Layers</th>
<th>Wire Area (mm²)</th>
<th>Composite Cross Section</th>
<th>Average Ultimate Load (kN)</th>
<th>Composite Strength* (MPa)</th>
<th>Wire Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mesh*</td>
<td>1</td>
<td>6.89</td>
<td>-</td>
<td>4.18</td>
<td>-</td>
<td>606</td>
</tr>
<tr>
<td>1LWM</td>
<td>1</td>
<td>6.89</td>
<td>1600</td>
<td>3.86</td>
<td>2.41</td>
<td>560</td>
</tr>
<tr>
<td>2LWM</td>
<td>2</td>
<td>13.79</td>
<td>1600</td>
<td>8.92</td>
<td>5.58</td>
<td>647</td>
</tr>
<tr>
<td>3LWM</td>
<td>3</td>
<td>20.68</td>
<td>1600</td>
<td>12.43</td>
<td>7.77</td>
<td>601</td>
</tr>
</tbody>
</table>

Note: *Composite strength determined by the ultimate load divided by the cross section of plates (Naaman 2000)

*Mesh = mesh coupon

As shown in Table 6.3, the ultimate load of mesh coupons is 8.3% higher than that of the ferrocement plates with one layer of wire mesh. The strength of wires for the ferrocement plates with one layer, two layers and three layers of wire mesh was 92%, 560
107% and 99% of the mesh coupon, respectively. Table 6.3 indicates that in tension the load carrying capacity of ferrocement composite is mainly depending on the mesh reinforcement. With the increase of the number of wire mesh layers, the ultimate tensile load increased. The mortar compressive strength did not affect the ultimate load of the composite. The mortar strength of each batch will be discussed shortly in Section 6.6.

It is reasonable to estimate the load carrying capacity of ferrocement in direction tension based on the product of the strength of reinforcement and the cross-sectional area of mesh reinforcement in the direction considered, shown as the following equation (ACI 549.1R-93):

\[ T = f_{uw} A_w \]  

(6.3)

where \( T \), \( f_{uw} \), and \( A_w \) are the tensile strength (load) of ferrocement composite, the ultimate stress of wires in the direction considered, the cross sectional area of wires in the direction considered, respectively. It is noted that for Eq. 6.3, \( A_w \) is the total area (only for this case) of the single wires in the direction considered.

The contribution of mortar matrix for the direct tensile strength is negligible (ACI 549R 1997), despite the essential contribution of matrix in compression. This was approved by the test results of the ferrocement plates. As mentioned earlier, the matrices were modified with EMACO 157 (manufactured by BASF). The compressive strength of the six batches of mortar was different. On the other hand, it is noted that if the matrix consists significant amount of epoxy or other kind of materials, which exhibits significant high tensile strength, the contribution of matrix may need to be taken into account (Ho et al. 2013).

Recall that the ultimate stress of single lateral wire was 620 MPa. In general, the estimated load carrying capacity was close to the experiment result, especially for the plates with three layers of wire mesh. Refer to Table 6.4 (the 3rd and 4th column) for details. Alternatively, the ultimate tensile load of ferrocement can be estimated based on the load carrying capacity of mesh coupon. The estimated load of the ferrocement composite was simply calculated by multiplying the average ultimate load of mesh coupon with the corresponding number of mesh layers. The estimation was close to
the experimental results, again particularly for the specimens with three layers of wire mesh. The method based on the mesh coupon had a slightly better agreement. Refer to Table 6.4 for details. The experimental results and the estimated values based on the two methods are illustrated in Fig. 6.10.

Table 6.4 Estimated tensile strength of ferrocement composite

<table>
<thead>
<tr>
<th>No. of Mesh Layers</th>
<th>Experiment Load T_{exp} (kN)</th>
<th>Estimated Load (Eq. 6.3) T_{est} (kN)</th>
<th>\frac{(T_{est}-T_{exp})}{T_{est}} (%)</th>
<th>Estimated Load* T_{est} (kN)</th>
<th>\frac{(T_{est}-T_{exp})}{T_{est}} (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.86</td>
<td>4.27</td>
<td>10.78</td>
<td>4.18</td>
<td>8.25</td>
</tr>
<tr>
<td>2</td>
<td>8.92</td>
<td>8.55</td>
<td>-4.19</td>
<td>8.35</td>
<td>-6.37</td>
</tr>
<tr>
<td>3</td>
<td>12.43</td>
<td>12.82</td>
<td>3.11</td>
<td>12.53</td>
<td>0.76</td>
</tr>
</tbody>
</table>

Note: * estimated based on the results of mesh coupon

Fig. 6.10 Experimental and estimated tensile strength of ferrocement composite

6.4 Tensile Strength Test of Steel Reinforcement

The tensile strength testing of steel reinforcement was conducted on the straight bars of the same batch of steel. The mechanical property of the steel can be changed by turning into a hoop (Pessiki et al. 2001). It is noted that the effect on the mechanical properties of steel due to manufacturing or sample preparation is not considered in this study.

The steel tensile strength test was conducted to investigate the actual strength and the stress-strain behaviour. Four specimens with an average length of 454 mm and an average diameter of 6.29 mm were tested under the monotonic tensile load until failure, complying with the Australian Standard AS 1391-2007. The procedure to determine the average diameter was the same as that for single wire strands. The tests
were conducted under displacement control using Instron 8033, which has a capacity of 500 kN. The test rate was mainly 1 mm/min. The applied load and the corresponding displacement were recorded by a data acquisition system that was connected to the machine. The gauge length was 291 mm. The average measured diameter of steel bars was 6.29 mm. The tensile strength was calculated by dividing the maximum applied load with the actual cross section area. The stress and strain curves of the four specimens are illustrated in Fig. 6.11 and the results are shown in Table 6.5.

![Stress and strain of steel bars](image)

**Fig. 6.11 Stress and strain of steel bars**

<table>
<thead>
<tr>
<th>Diameter (mm)</th>
<th>Yield Stress (MPa)</th>
<th>Ultimate Stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Average Standard Deviation</td>
<td>Average Standard Deviation</td>
</tr>
<tr>
<td>6.29</td>
<td>657.7 13.7</td>
<td>697.8 6.8</td>
</tr>
</tbody>
</table>

As shown in Fig. 6.11, no clear difference appeared between the yield stress and the ultimate stress and strain hardening did not occur. Offset yield stress based on a standard strain of 0.2%. According to Sheikh and Uzumeri (1980), the stress-strain characteristics of steel reinforcement greatly affect the situation of the confining pressure. The upper boundary of the confining pressure was governed by the yield strength of lateral steel reinforcement. Fig. 6.11 shows once steel yields, the stress turned constant for a huge range of strain, which indicates that steel reinforcement
became ineffective in constraining the expansion of concrete core (Mirmiran and Shahawy 1997).

Furthermore, compared to the steel reinforcement with relatively lower yield stress (refer to Fig. 4.2 in Chapter 4), the confining pressure, provided by the relatively high strength steel reinforcement, continued to increase with increased strain well beyond the strains at which the former steel reinforcement would have yielded (Pessiki et al. 2001).

6.5 Properties of Normal Strength Concrete
The tests of fresh concrete and hardened concrete were focused on workability and the compressive strength. The concrete had a slump of 175 mm. The test was conducted as indicated in the previous experiments.

6.5.1 Concrete Compressive Strength Test
The concrete was cast in the same way as indicated in Section 3.2. Refer to Chapter 4 for concrete curing. The average water temperature was 23ºC. Three concrete cylinders with 100 mm in diameter and 200 mm in height were scheduled to test for the compressive strength at 7 days, 28 days and 56 days. In addition, two concrete cylinders with 150 mm in diameter and 300 mm in height were scheduled to test for the compressive strength at 7 days, 28 days and 56 days.

The compressive strength tests were conducted in a similar way as indicated in Chapter 4. However, two types of load control rates were applied: for Ø100 × 200 mm cylinder, the load was applied as 17.5% of 360 kN, which was equivalent to 0.13 MPa/s, while for Ø150 × 300 mm cylinder, the load was applied as 40% of 900 kN, equivalent to 0.34 MPa/s. The results of the compressive strength of standard cylinders with respect to age and size are shown in Table 6.6 and illustrated in Fig. 6.12.
Table 6.6 Compressive strength of standard cylinders

<table>
<thead>
<tr>
<th>Sample</th>
<th>Age (Days)</th>
<th>Diam. D (mm)</th>
<th>Height H (mm)</th>
<th>Mass (kg)</th>
<th>Max Load (kN)</th>
<th>Strength f'c (MPa)</th>
<th>Average Strength f''c (MPa)</th>
<th>Fracture Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7</td>
<td>101.5</td>
<td>202</td>
<td>3.85</td>
<td>193.5</td>
<td>23.9</td>
<td>22.1</td>
<td>I</td>
</tr>
<tr>
<td>2</td>
<td>7</td>
<td>101.5</td>
<td>202</td>
<td>3.86</td>
<td>167.8</td>
<td>20.7</td>
<td>III</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>7</td>
<td>101.0</td>
<td>203</td>
<td>3.86</td>
<td>174.2</td>
<td>21.8</td>
<td>I</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>28</td>
<td>101.0</td>
<td>201</td>
<td>3.86</td>
<td>262.0</td>
<td>32.7</td>
<td>III</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>28</td>
<td>99.5</td>
<td>200.5</td>
<td>3.86</td>
<td>257.0</td>
<td>33.1</td>
<td>I</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>28</td>
<td>101.5</td>
<td>201.5</td>
<td>3.84</td>
<td>270.0</td>
<td>33.4</td>
<td>I</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>56</td>
<td>100.0</td>
<td>199.8</td>
<td>3.69</td>
<td>275.0</td>
<td>35.1</td>
<td>35.2</td>
<td>I</td>
</tr>
<tr>
<td>2</td>
<td>56</td>
<td>100.0</td>
<td>199.5</td>
<td>3.67</td>
<td>277.5</td>
<td>35.4</td>
<td>I</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>7</td>
<td>149.9</td>
<td>299.5</td>
<td>12.31</td>
<td>358.0</td>
<td>20.3</td>
<td>19.6</td>
<td>*</td>
</tr>
<tr>
<td>2</td>
<td>7</td>
<td>149.5</td>
<td>300.0</td>
<td>12.35</td>
<td>333.0</td>
<td>19.0</td>
<td>I</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>28</td>
<td>151.0</td>
<td>302.0</td>
<td>12.60</td>
<td>497.5</td>
<td>27.8</td>
<td>28.1</td>
<td>I</td>
</tr>
<tr>
<td>2</td>
<td>28</td>
<td>150.2</td>
<td>301.3</td>
<td>12.30</td>
<td>502.5</td>
<td>28.4</td>
<td>I</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>56</td>
<td>151.0</td>
<td>299.5</td>
<td>12.92</td>
<td>561.0</td>
<td>31.3</td>
<td>31.7</td>
<td>I</td>
</tr>
<tr>
<td>2</td>
<td>56</td>
<td>149.9</td>
<td>299.8</td>
<td>12.38</td>
<td>565.0</td>
<td>32.0</td>
<td>I</td>
<td></td>
</tr>
</tbody>
</table>

Note:
Max. = Maximum; * Not recorded
Refer to C39/C39M-10, ASTM for the fracture modes.
Type I: well-formed cones on both ends; Type III: vertical cracking through both ends.

Fig. 6.12 Concrete compressive strength

As shown in the table and figure above, the compressive strength of Ø100 × 200 mm cylinders was higher than that of Ø150 × 300 mm cylinders. However, cylinders with
two different types of sizes exhibited a similar trend that at early stage (within 7 days) the strength increase rate was significantly faster than that at later stages (28 days and 56 days). At 7 days, the compressive strength reached 69.2% of the specified strength (32 MPa) for Ø100 × 200 mm cylinders, 61.4% for Ø150 × 300 mm cylinders. Comparing with the strength at Day 7, the strength at 28 days increased 49.3% and 43.1%, respectively, for Ø100 × 200 mm and Ø150 × 300 mm cylinders; the strength at 56 days improved 59.1% and 61.3%, respectively, for Ø100 × 200 mm and Ø150 × 300 mm cylinders.

6.5.2 Effect of Specimen Size

Fig. 6.13 indicates the effect of the specimen size on the compressive strength. The compressive strength of Ø150 × 300 mm cylinders at 7 days, 28 days and 56 days was 89%, 85% and 90%, respectively, of Ø100 × 200 mm cylinders. This observation is consistent with the suggestion by other researchers (Martinez et al. 1984) that the ratio of compressive strength of the larger to the smaller cylinders to be close to 0.90.

6.5.3 Concrete Static Modulus of Elasticity in Compression

The tests were conducted in a similar way as indicated in Section 3.13. For each specimen, two 60 mm strain gauges were placed vertically opposite each other. The tests were conducted using Instron 8033. Most published empirical formulas for the static elastic modulus of concrete are related to the characteristic compressive strength at 28 days and the surface dry unit weight of the concrete (Attard and
Based on the work of Pauw (1960), Eq. 6.4 is recommended in ACI 318-08 for NSC with density ranged between 1440 and 2560 kg/m$^3$. This formula is also adopted by AS 3600 (2009) for determining concrete elastic modulus with mean value of the in situ compressive strength of concrete not greater than 40 MPa. According to ACI 318-08, for normal weight concrete, modulus of elasticity is determined by Eq. 6.5.

\[
E_c = 0.043 \rho^{1.5} \sqrt{f'_c} \quad (6.4)
\]

\[
E_c = 4700 \sqrt{f'_c} \quad (6.5)
\]

where $\rho$ is the surface dry unit weight in kg/m$^3$; $f'_c$ is the characteristic compressive (cylinder) strength of concrete at 28 days. It is recognised that Eq. 6.4 is only approximately and accurate to within about 20 percent for NSC (Attard and Setunge, 1996). The experimental results and the estimated modulus according to Eq. 6.4 and Eq. 6.5 are summerised in Table 6.7.

<table>
<thead>
<tr>
<th>Experiment</th>
<th>Density $\rho$ (kg/m$^3$)</th>
<th>Estimation $E_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Eq. 6.4</td>
</tr>
<tr>
<td>27089</td>
<td>2335</td>
<td>27918</td>
</tr>
</tbody>
</table>

As shown in Table 6.7, the two methods provided close results. The compressive stress and strain curves are shown in Fig. 6.14. The specimens were tested to failure after the cyclic loading. The average compressive strength after modulus test was 35.5 MPa. Each specimen exhibited Type I failure mode (C39/C39M-10, ASTM).
6.6 Properties of Modified High Strength Mortar

This section covers the trial mortar mixes and the final mortar mixes for the confined specimens, with an emphasis on the mix design and mortar properties. The discussion on the test results is focused on the compressive strength, the flexural strength and the flow ability.

6.6.1 Mortar Trial Mixes

According to (Naaman 2000), matrix of ferrocement has huge influence on the behaviour of the final product. Intensive trials on mortar mixes were conducted. It was aimed to find the proper proportion of water, cement, sand and fly ash in terms of strength, consistency and workability. Other performance criteria such as impermeability, sulphate resistance and corrosion protection were not considered in the present study. The compressive strength of the matrices ranged from 48 MPa (28 days) to 69 MPa (91 days). It is noted that due to equipment constraint, some of the testing had to be postponed.

Superplasticizer (POZZOLITH 370 PC, made by BASF) was used for all the mixes. Water content of sand was investigated for each casting and the water content was reflected into the total amount of water used. At least three prism specimens with a
dimension of 40 mm × 40 mm × 160 mm were prepared for each trial mix. Before casting the steel mould was evenly greased with mechanic oil. Mortar was placed into the steel moulds in two layers. The steel mould had three identical compartments. A shaking table was used to facilitate compacting mortar. After compacting a small amount of mortar (of the same batch) might be added for making the surface even. At each casting flow table test was conducted in situ to investigate consistency and workability. The flow table test was ducted in complying with ASTM standard C 230/C 230M (2008) and ASTM standard C1437 (2007). Shaking table was used to compact mortar mix. Four hours after casting specimens were transferred into the moisture room set at 22°C and 50% relative humidity content. After maintained at the moisture room for 24 - 48 hours depending on the degree of mortar hardening, specimens were removed from mould and placed in the water tank for curing.

Two types of sand were used: fine sand and relative coarse sand. The second type of sand was adopted for most of the trials and the final castings for the confined specimens. Refer to Table 6.8 for the details of the fineness modulus of the relative coarse sand, passing the number 8 sieve. According to the fineness modulus the second type of sand can be classified as medium sand. Most of the mortar trial mixes are summerised in Table 6.9.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Weight of sand Retained (g)</th>
<th>Cumulative Weight of sand retained (g)</th>
<th>Cumulative Percentage of sand Retained (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.36 mm (No.8)</td>
<td>156.7</td>
<td>156.7</td>
<td>13</td>
</tr>
<tr>
<td>1.18 mm (No.16)</td>
<td>147.8</td>
<td>304.5</td>
<td>26</td>
</tr>
<tr>
<td>600 μm (No.30)</td>
<td>238.3</td>
<td>542.8</td>
<td>46</td>
</tr>
<tr>
<td>300 μm (No.50)</td>
<td>384.9</td>
<td>927.7</td>
<td>79</td>
</tr>
<tr>
<td>150 μm (No.100)</td>
<td>185.5</td>
<td>1113.2</td>
<td>95</td>
</tr>
<tr>
<td>Left</td>
<td>59.8</td>
<td>1173.0</td>
<td>-</td>
</tr>
<tr>
<td>Total</td>
<td>1173</td>
<td>4217.9</td>
<td>260</td>
</tr>
</tbody>
</table>

Note: Fineness Modulus =260/100=2.6
Table 6.9 Trial mortar mixes

<table>
<thead>
<tr>
<th>Batch Code</th>
<th>Mix Design (by weight of cement)</th>
<th>Cement</th>
<th>Medium Sand</th>
<th>Fine Sand</th>
<th>Water</th>
<th>Fly ash</th>
<th>Superplasticiser (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MB1</td>
<td>1</td>
<td>2.0</td>
<td>0.41</td>
<td>0.21</td>
<td></td>
<td></td>
<td>2.0</td>
</tr>
<tr>
<td>MB2</td>
<td>1</td>
<td>2.5</td>
<td>0.50</td>
<td>0.20</td>
<td></td>
<td></td>
<td>2.9</td>
</tr>
<tr>
<td>MB3</td>
<td>1</td>
<td>2.5</td>
<td>0.50</td>
<td>0.20</td>
<td></td>
<td></td>
<td>2.0</td>
</tr>
<tr>
<td>MB4</td>
<td>1</td>
<td>2.5</td>
<td>0.52</td>
<td>0.15</td>
<td></td>
<td></td>
<td>2.0</td>
</tr>
<tr>
<td>MB5</td>
<td>1</td>
<td>2.0</td>
<td>0.41</td>
<td>0.15</td>
<td></td>
<td></td>
<td>2.0</td>
</tr>
<tr>
<td>MB6</td>
<td>1</td>
<td>2.0</td>
<td>0.45</td>
<td>0.15</td>
<td></td>
<td></td>
<td>2.0</td>
</tr>
<tr>
<td>MB7</td>
<td>1</td>
<td>1.95</td>
<td>0.38</td>
<td>0.15</td>
<td></td>
<td></td>
<td>2.5</td>
</tr>
<tr>
<td>MB8</td>
<td>1.98</td>
<td></td>
<td>0.42</td>
<td>0.17</td>
<td></td>
<td></td>
<td>2.0</td>
</tr>
<tr>
<td>MB9</td>
<td>2.50</td>
<td></td>
<td>0.39</td>
<td>0.15</td>
<td></td>
<td></td>
<td>2.4</td>
</tr>
<tr>
<td>MB10</td>
<td>2.50</td>
<td></td>
<td>0.36</td>
<td>0.15</td>
<td></td>
<td></td>
<td>2.7</td>
</tr>
<tr>
<td>MB11</td>
<td>2.50</td>
<td></td>
<td>0.36</td>
<td>0.16</td>
<td></td>
<td></td>
<td>2.0</td>
</tr>
<tr>
<td>MB12</td>
<td>2.48</td>
<td></td>
<td>0.48</td>
<td>0.15</td>
<td></td>
<td></td>
<td>2.1</td>
</tr>
<tr>
<td>MB13</td>
<td>2.51</td>
<td></td>
<td>0.38</td>
<td>0.15</td>
<td></td>
<td></td>
<td>1.9</td>
</tr>
<tr>
<td>MB14</td>
<td>2.47</td>
<td></td>
<td>0.40</td>
<td>0.15</td>
<td></td>
<td></td>
<td>2.0</td>
</tr>
<tr>
<td>MB15</td>
<td>2.49</td>
<td></td>
<td>0.40</td>
<td>0.15</td>
<td></td>
<td></td>
<td>2.0</td>
</tr>
<tr>
<td>MB16</td>
<td>2.47</td>
<td></td>
<td>0.42</td>
<td>0.16</td>
<td></td>
<td></td>
<td>1.9</td>
</tr>
</tbody>
</table>

6.6.2 Compressive and Flexural Strength of Mortar

The results of mortar mixes MB1-MB16 are presented in Table 6.10, including the density and the flow table test results. The tests of the trial mortar mixes were conducted by Boral Material Services Laboratory.

Table 6.10 Results of trial mortar mixes

<table>
<thead>
<tr>
<th>Mix Code</th>
<th>Age (days)</th>
<th>Density (kg/m³)</th>
<th>Compressive Strength (MPa)</th>
<th>Flexural Strength (MPa)</th>
<th>Flow (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MB1</td>
<td>91</td>
<td>2112</td>
<td>68.0</td>
<td>-</td>
<td>87</td>
</tr>
<tr>
<td>MB2</td>
<td>91</td>
<td>2082</td>
<td>49.9</td>
<td>-</td>
<td>73</td>
</tr>
<tr>
<td>MB3</td>
<td>91</td>
<td>2055</td>
<td>51.3</td>
<td>9.1</td>
<td>53</td>
</tr>
<tr>
<td>MB4</td>
<td>91</td>
<td>2095</td>
<td>59.5</td>
<td>9.4</td>
<td>74</td>
</tr>
<tr>
<td>MB5</td>
<td>91</td>
<td>2148</td>
<td>69.3</td>
<td>10.1</td>
<td>43</td>
</tr>
<tr>
<td>MB6</td>
<td>14</td>
<td>2167</td>
<td>32.9</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>MB7</td>
<td>28</td>
<td>2108</td>
<td>51.6</td>
<td>9.4</td>
<td>-</td>
</tr>
<tr>
<td>MB8</td>
<td>14</td>
<td>2248</td>
<td>54.4</td>
<td>18.8</td>
<td>-</td>
</tr>
<tr>
<td>MB9</td>
<td>14</td>
<td>2270</td>
<td>57.1</td>
<td>18.8</td>
<td>117</td>
</tr>
<tr>
<td>MB10</td>
<td>14</td>
<td>2283</td>
<td>64.4</td>
<td>11.8</td>
<td>77</td>
</tr>
<tr>
<td>MB11</td>
<td>14</td>
<td>2294</td>
<td>67.9</td>
<td>10.2</td>
<td>57</td>
</tr>
<tr>
<td>MB12</td>
<td>14</td>
<td>1952</td>
<td>19.1</td>
<td>5.6</td>
<td>50</td>
</tr>
<tr>
<td>MB13</td>
<td>14</td>
<td>2225</td>
<td>57.5</td>
<td>9.5</td>
<td>89</td>
</tr>
<tr>
<td>MB14</td>
<td>28</td>
<td>2188</td>
<td>48.0</td>
<td>10.7</td>
<td>112</td>
</tr>
<tr>
<td>MB15</td>
<td>28</td>
<td>2207</td>
<td>53.4</td>
<td>9.9</td>
<td>91</td>
</tr>
<tr>
<td>MB16</td>
<td>28</td>
<td>2238</td>
<td>57.5</td>
<td>10.6</td>
<td>76</td>
</tr>
</tbody>
</table>

Note: – not available or not recorded.
The test results of mortar mixes MB1-MB5 are presented in Fig. 6.15. Due to the equipment constraint, the tests were conducted on 91 days. Fine sand was used for these batches. The flexural results were not available for MB1 and MB2.

As shown in Fig. 6.15, the water content had significant influence on the compressive strength. The content of fly ash also affects the compressive strength. Due to its lowest w/c ratio and the least fly ash content, MB5 had the highest compressive strength, closely followed by MB5. Among MB2, MB3 and MB4, with the decrease in fly ash and slight increase in water content, the compressive strength increased significantly. On the other hand, the results intended to show that for a relatively long term, the flexural strength was not that sensitive to the change in the proportion of materials since the flexural strength varied slightly contrast to the large change in the compressive strength.

The test results of mortar mixes MB6, MB8, MB9, MB10, MB11, MB12 and MB13 are presented in Fig. 6.16. The tests were conducted on 14 days. Corse sand was adopted, except for MB12. The flexural strength of MB6 was not available.
The figure above indicates that the compressive strength of matrix is largely governed by the water-cement ratio. By contrast, the flexural strength did not necessarily reverse to the water-cement ratio. MB11 and MB10 had the first and second highest compressive strength due to relatively low water-cement ratio, but the corresponding flexural strength was much less than that of MB8 and MB9. MB12 had the lowest value in terms of both the compressive strength and the flexural strength. This was likely caused by the highest water content and particle size. It is note that MB8 appeared rather liquid, which is likely due to the less sand-cement ratio and relatively high water content. However, MB8 had the highest value in flexural strength.

One of the challenges associated with applying normal mortar is about penetrating mesh apertures and ensuing good bonding condition. Wet agent EMACO 157 (manufactured by BASF) was used to improve the bonding condition, plastic consistency and workability for mortar mixes MB14-MB16. The variable in these three mixes was the proportion of blending EMACO 157 into water. The total weight of EMACO 157 and water was calculated as water added into the mix. The ratio of water and EMACO 157 was 1:1, 2:1 and 2.5:1 by weight, respectively, for mortar mixes MB14, MB15 and MB16. Corse sand was adopted and the tests were conducted on 28 days. The test results of mortar mixes MB14-MB16 are presented in Fig. 6.17.
Fig. 6.17 indicates that the compressive strength decreased with the increased proportion of EMACO157; while on the other hand, it seemed that the flexural strength was not affected by the change of EMACO157 proportion. The flow table results of MB14-MB16 show that EMACO157 can improve the flow ability.

The relationship between the compressive and flexural strength is illustrated in Fig. 6.18 and Fig. 6.19. It seems that the ratio of the flexural and compressive strength fluctuates approximately between 13% and 22% for most of the batches, except for MB8, MB9 and MB12. The ratio of these mixes was 34.6%, 32.9% and 29.2%, respectively. MB12 had the lowest compressive strength. This is likely because of the highest water-cement ratio. In addition, fine sand was adopted for MB12 among mortar casting MB8-MB13. However, in the long term a relatively high compressive strength can be achieved despite relatively high water-cement ratio and find sand as shown by MB1-MB5.
6.6.3 Final Mortar Mixes for Confined Specimens

Recall that the specimens were divided into three groups and each group consisted of two replicates for each category. Due to the equipment constraints, two batches of mortar were cast for each group. In total six batches of mortar cast, as shown in Table 6.11. EMACO 157 (manufactured by BASF) was adopted for the mixing. The ratio of water and EMACO 157 was 2.5:1 by weight. Mortar casting FMB1 was used for the first replicate of Group One Specimens, including Specimens MC1, 1M0S1, 1M100S1, 1M75S1 and 1M50S1; FMB2 was applied to the second replicate of Group One Specimens, including Specimens MC2, 1M0S2, 1M100S2, 1M75S2 and 1M50S2. The application of the remaining batches followed a similar pattern.

Table 6.11 Mortar mixes for confined specimens

<table>
<thead>
<tr>
<th>Mortar Batch</th>
<th>Mix Design (by weight of cement)</th>
<th>Confined Specimens Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>FMB1</td>
<td>1 2.0 0.34 0.15 1.9</td>
<td>I</td>
</tr>
<tr>
<td>FMB2</td>
<td>1 2.0 0.39 0.15 2.3</td>
<td>I</td>
</tr>
<tr>
<td>FMB3</td>
<td>1 2.0 0.34 0.15 1.8</td>
<td>II</td>
</tr>
<tr>
<td>FMB4</td>
<td>1 2.0 0.34 0.15 1.7</td>
<td>II</td>
</tr>
<tr>
<td>FMB5</td>
<td>1 2.0 0.34 0.15 1.9</td>
<td>III</td>
</tr>
<tr>
<td>FMB6</td>
<td>1 2.0 0.34 0.15 1.8</td>
<td>III</td>
</tr>
</tbody>
</table>

For each mortar casting, a number of Ø100 × 200 mm cylinders were cast for the compressive strength test. It is noted that extra time was needed for hydration before demoulding. The curing was the same as indicated in Section 6.5.1. The flow table...
tests were conducted for FMB1-FMB6. Refer to Section 6.4.1 for the details. Fig. 6.20 provides an image of the mortar flow ability. The compressive strength tests were conducted as indicated in Section 6.5.1 (under the lower load control rate). The test results of FMB1-FMB6 are summerised in Table 6.12 and illustrated in Fig. 6.21, which indicate that the mortar matrices had not only high strength but also good flow ability.

![Fig. 6.20 Flow table test of FMB3](image)

**Table 6.12 Test results of the final mortar casting**

<table>
<thead>
<tr>
<th>Mix Code</th>
<th>Compressive Strength</th>
<th>Average Age (Days)</th>
<th>Average (MPa)</th>
<th>Average Density (kg/m³)</th>
<th>Average Flow (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FMB1</td>
<td>28</td>
<td>62.1</td>
<td>2229</td>
<td>82</td>
<td></td>
</tr>
<tr>
<td>FMB2</td>
<td>28</td>
<td>48.5</td>
<td>2153</td>
<td>136</td>
<td></td>
</tr>
<tr>
<td>FMB3</td>
<td>28</td>
<td>59.5</td>
<td>2226</td>
<td>87</td>
<td></td>
</tr>
<tr>
<td>FMB4</td>
<td>28</td>
<td>55.6</td>
<td>2180</td>
<td>90</td>
<td></td>
</tr>
<tr>
<td>FMB5</td>
<td>29</td>
<td>61.6</td>
<td>2162</td>
<td>121</td>
<td></td>
</tr>
<tr>
<td>FMB6</td>
<td>28</td>
<td>63.5</td>
<td>2198 *</td>
<td>*</td>
<td></td>
</tr>
</tbody>
</table>

*Note: *FMB6 was mixed twice and fully mixed at the second time. The flow table test result was not recorded during the process.*
The above figure and table show that the compressive strength of FMB1-FMB6 varied from 48.5 MPa to 63.5 MPa. Mortar batch FMB2 had the lowest compressive strength mainly due to obviously relative high water-cement ratio. The variation on the compressive strength with the same water-cement ratio is possibly due to: 1) the scale of the mixing was considerably large, compared to the trial mixes using the mixing bowl; 2) for the mortar mix, the proportion of sand is large; 3) the samples for the water content test might not fully reflect the condition of the total amount of sand; 4) just as the properties of concrete tend to vary (Guo and Shi 2003), a similar situation can occur with mortar. Therefore, in order to investigate the effect of mortar matrices on the behavior of confined specimens, it is important to determine the compressive strength of mortar for each casting.

6.7 Summary

The properties of constituent materials influence the behaviour of the composite and the members strengthened with the composite. In this chapter the preliminary testing results mainly consist of 1) the tensile strength of steel reinforcement, single wire and wire mesh reinforcement; 2) the tensile load-elongation response of ferrocement plates; 3) the compressive strength of concrete and mortar, also including the mortar trials; and 4) the elasticity modulus of concrete. The discussion of results for each type of testing has been provided. A detailed approach to achieve both high strength and flow able mortar matrices was developed through practice. The next chapter will present the testing for the confined specimens and the test results discussion.
CHAPTER 7 EXPERIMENT III TEST RESULTS AND ANALYSIS

7.1 Introduction
In this Chapter, firstly, the failure modes are represented; secondly, the results are shown for each group with the emphasis on the relationship of the axial load and axial displacement, ductility and energy absorption; and thirdly, the experimental ultimate load is compared with a number of existing models and the proposed models.

7.2 Testing of Group One Specimens
To investigate the effectiveness of the steel hoops, wire mesh and modified high strength mortar (MHSM) composite, a total of 38 specimens were prepared and tested, among which six were plain concrete (Ø150 mm × 380 mm); 32 specimens were confined specimens with various amounts of transverse reinforcement (Ø190 × 380 mm). Refer to Section 5.2 for the configuration details. All the specimens were tested by displacement controlled concentric loading, as indicated in Chapter 5.

Each of the first two groups consisted of six categories: plain concrete; MHSM confined specimens; MHSM and wire mesh strengthened specimens; and specimens strengthened with MHSM, wire mesh and steel hoops at three different spacing (100 mm, 75 mm and 50 mm). Mortar FMB1 and FMB2 were applied to the first and the second replicates of Group One Specimens, respectively.

7.2.1 Failure Modes
For the convenience of comparison, the failure modes of MHSM confined specimens as well as wire mesh and MHSM confined specimens of the three groups are summarised in the following section. The failure modes for the steel hoops, wire mesh and MHSM confined specimens will be introduced group by group.
For MHSM confined specimens, tiny visible cracks initiated randomly on the mortar circumference at about 30% of the peak load. With the increase of load, cracks increased in number, length and width. As the ultimate load was being reached, the existing cracks lengthened along the length of the specimens and divided the cover into long sections while the width of crack widened. However, the mortar shell did not spall, as shown in Fig. 7.1 (a). Past the ultimate load, cracking developed further and mortar shell bulged at the mid height with large pieces of mortar spalled, as shown in Fig. 7.1 (b). Well-formed core exhibited on both ends after removing the spalled cover pieces (AMTS C39/C39M-10), as shown in Fig. 7.1 (c). During the after-test investigation some of the specimens collapsed, as shown in Fig. 7.1 (d).

For wire mesh and MHSM confined specimens, tiny vertical cracks appeared at the mid height to the top of the specimen. Tiny visible vertical cracks appeared as early as approximately 40% of the peak load. Lateral cracks appeared later. As the test proceeded, cracks increased in terms of numbers, length and width. Past the peak load, the vertical cracks widened significantly and mortar cover spalled at the mid height around the circumference. A couple of single wires snapped when approaching to the ultimate load, which was likely because those single wires ruptured earlier while the majority of wires did not reached the yield strength. This was consistent with the observation during the tensile strength test of mesh coupons. Wires snapped continuously past the ultimate load. The investigation showed that lateral wires ruptured in tension and vertical wires buckled, evidencing the confinement contributed by wire mesh. Typical vertical cracking suggested tension
failure in the wire mesh and mortar composite. It is noted that diagonal pattern was also observed on the wire mesh and mortar tube. The selected specimens at different stages are shown in Fig. 7.2-Fig. 7.4.

Fig. 7.2 Wire mesh confined specimens past the ultimate load

Fig. 7.3 Wire mesh confined specimens at the end of test

Fig. 7.4 Tested wire mesh confined specimens

SG=strain gauge
Fig. 7.2-Fig. 7.4 show that with the presence of wire mesh, MHSM cover behaved differently from HSC. For Specimens 1M0S1 and 1M0S2, part of cover became loose but did not spall away at the end of test; for Specimens 3M0S1 and 3M0S2, mortar cover spalled in a gradual manner. It was observed that strain gauges on the core were pushed towards the wire mesh tube. As shown in Fig. 7.3, with the increase in the number of wire mesh layers, MHSM cover spalled more.

As suggested (Cusson and Paultre 1994), between the core and the cover, a weakness plane may form due to a large amount of lateral reinforcement. However, according to Foster et al. (1998), cover spalling is the result of the difference in restraint of the Poisson growth between the core and cover. Under concentric loading, the dilation of concrete core is constrained by passive confinement, while the cover shell is constrained by the tensile stress at the cover and core interface. Once the tensile stress across the interface is greater than the tensile capacity of concrete, a cracking plane created. For the case of composite tube comprising steel, wire mesh and different matrices (compared to the concrete core), it is more complicated and the interface between the outmost layer of wire mesh and the mortar protective cover is critical for cover spalling.

For each category of the steel hoops, wire mesh and MHSM confined specimens, a general trend was observed that with the increase of the mesh layers, cover spalled severe and the deformation in the reinforcement composite and the concrete core became more considerable. The selected specimens of Group One at the end of test and at investigation are shown in Fig. 7.5 and Fig. 7.6, respectively.
Within the specimens confined with one layer of wire mesh, steel hoops and MHSM, vertical cracking was again the dominated phenomenon on the cover, despite the appearance of some lateral cracks. Tiny visible vertical cracks appeared as early as about 40-50% of the ultimate load. As the test proceeded, cracks extended and increased in width and numbers. Accordingly, small and thin pieces of mortar cover peeled.

Past the ultimate load, cover started bulging and gradually peeled off. Wires started snapping mainly past the ultimate load, although in some case wire snapped when approaching to the ultimate load. Vertical cracks on the wire mesh and mortar tube widened considerably. Strain gauges originally placed on steel hoops were pushed onto the wire mesh tube due to the dilation of concrete core. At the late stage of test, steel hoops snapped at the end of welded area, with a loud sound. Mortar cover almost spalled off completely and the failure mode of concrete cores exhibited in the shape of double cone (AMTS C39/C39M-10) at the end of test. Refer to Fig. 7.6(a), (b) and (c) for details.
7.2.2 Axial Load and Axial Displacement of Group One Specimens

The axial load and displacement curves of the first group are illustrated in Fig. 7.7 and Fig. 7.8, respectively, for the first and second replicates. The test results are presented in Table 7.1 and Table 7.2. Due to operation error, the testing data for the first group was partially recorded past the ultimate load.
### Table 7.1 Test results of Group One Specimens_1

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Volumetric Load</th>
<th>Yield Load</th>
<th>Ultimate Load</th>
<th>Peak Load 1st (kN)</th>
<th>Peak Load 2nd (kN)</th>
<th>Axial Displacement at $P_u$ ($mm$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FTC1</td>
<td>644</td>
<td>667</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.36</td>
</tr>
<tr>
<td>MC1</td>
<td>1156</td>
<td>1184</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.31</td>
</tr>
<tr>
<td>1M0S1</td>
<td>0.21</td>
<td>1036</td>
<td>1086</td>
<td>1086</td>
<td>-</td>
<td>1.48</td>
</tr>
<tr>
<td>1M100S1</td>
<td>0.91</td>
<td>948</td>
<td>1009</td>
<td>1009</td>
<td>-</td>
<td>1.46</td>
</tr>
<tr>
<td>1M75S1</td>
<td>1.16</td>
<td>1039</td>
<td>1100</td>
<td>1100</td>
<td>-</td>
<td>1.29</td>
</tr>
<tr>
<td>1M50S1</td>
<td>1.64</td>
<td>1102</td>
<td>1176</td>
<td>1176</td>
<td>-</td>
<td>1.42</td>
</tr>
</tbody>
</table>

*Refer to Section 5.2.4 and 5.2.5 in Chapter 5 for the volumetric ratios of lateral reinforcement.

### Table 7.2 Test results of Group One Specimens_2

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Volumetric Load</th>
<th>Yield Load</th>
<th>Ultimate Load</th>
<th>Peak Load 1st (kN)</th>
<th>Peak Load 2nd (kN)</th>
<th>Axial Displacement at $P_u$ ($mm$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FTC2</td>
<td>-</td>
<td>606</td>
<td>627</td>
<td>-</td>
<td>-</td>
<td>1.41</td>
</tr>
<tr>
<td>MC2</td>
<td>-</td>
<td>1149</td>
<td>1194</td>
<td>-</td>
<td>-</td>
<td>1.60</td>
</tr>
<tr>
<td>1M0S2</td>
<td>0.21</td>
<td>1014</td>
<td>1064</td>
<td>1064</td>
<td>-</td>
<td>1.35</td>
</tr>
<tr>
<td>1M100S2</td>
<td>0.91</td>
<td>833</td>
<td>914</td>
<td>914</td>
<td>-</td>
<td>1.81</td>
</tr>
<tr>
<td>1M75S2</td>
<td>1.16</td>
<td>839</td>
<td>919</td>
<td>919</td>
<td>-</td>
<td>2.04</td>
</tr>
<tr>
<td>1M50S2</td>
<td>1.64</td>
<td>893</td>
<td>958</td>
<td>958</td>
<td>952</td>
<td>1.55</td>
</tr>
</tbody>
</table>

*Refer to Section 5.2.4 and 5.2.5 in Chapter 5 for the volumetric ratios of lateral reinforcement.

Fig. 7.8 Axial load-axial displacement of Group One Specimens_2
Recall that the mortar compressive strength for first and second replicate was 62.1 MPa and 48.5 MPa, respectively. No matter which batch of mortar was applied, the confined specimens with either wire mesh or steel hoop had lower ultimate load, compared to only mortar confined specimens (Specimens MC1 and MC2). However, the failure mode of Specimens MC1 and MC2 was brittle, evidenced by a sudden load drop shortly past the ultimate load.

Compared to their counterparts, the first replicates of each category presented a higher value in terms of the ultimate load. Specimens 1M0S1 and 1M100S1 were repaired due to surface imperfection. With the same amount of steel reinforcement volumetric ratio \( (\rho_s) \), Specimens 1M75S1 and 1M75S2, Specimens 1M50S1 and 1M50S2 exhibited significantly different slope on both the ascending and descending batches of the axial load and displacement curves. For Specimens 1M75S1 and 1M50S1, the load dropped sharply past the ultimate load. For Specimen 1M50S1, the load was picked up after dropping, evidenced by a significant change in the slope. This indicates that the confinement became activated. By contrast, Specimens 1M75S2 dropped gradually; Specimen 1M50S2 represented a load plateau, with a second peak load of 952 kN, almost as high as the first peak load. The long plateau indicated that MHSIM cover lost its load carrying capacity gradually, while simultaneously the passive confinement was gradually activated. The different behaviour among the first and second replicates was likely due to the difference in the compressive strength of mortar.

The results showed that when the lateral reinforcement ratio was applied at a low range from 0.21% to 1.64 %, the ultimate load was affected adversely. The slope of the axial load and displacement curves of the descending part is pronouncedly affected by the amount of steel reinforcement.

### 7.3 Testing of Group Two Specimens

For the specimens of Group Two, Mortar FMB3 (59.5 MPa) and FMB4 (55.6 MPa) were applied to the first and the second replicates, respectively. The failure modes and the axial load and displacement curves are shown below.
7.3.1 Failure Mode of Group Two Specimens

The select specimens at the end of tests are shown in Fig. 7.9. As mentioned earlier, the failure modes of each category were similar to their counterpart in Group One, but demonstrated a more considerable deformation. Past the peak load cover started spalling. For Specimens 2M50S1 and 2M50S2, while cover started spalling, the load was fluctuating. The cover of Specimen 2M75S1 spalled in a symmetric manner, as shown in (b) of Fig. 7.9. It was also observed that large pieces of mortar cover spalled. This is probably related to the increase in the amount of reinforcement. The spacing of cracks might be reduced by improving the distribution of mesh. As mentioned earlier in Chapter 6, the wire mesh layers were closed formed into reinforcement tube. The selected specimens of Group Two at investigation are shown in Fig. 7.10.

![Fig. 7.9 Group Two Specimens at the end of tests](image1)

![Fig. 7.10 Group Two Specimens at investigation](image2)
7.3.2 Axial Load and Axial Displacement of Group Two Specimens

The axial load and displacement curves of the first and second replicates of Group Two are illustrated in Fig. 7.11 and Fig. 7.12, respectively. The test results are presented in Table 7.3 and Table 7.4.

![Axial load- axial displacement of Group Two Specimens](image)

**Table 7.3 Test results of Group Two Specimens**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Reinforcement Volumetric Ratio* (%)</th>
<th>Yield $P_Y$ (kN)</th>
<th>Ultimate $P_u$ (kN)</th>
<th>Peak Load $1^{st}$ (kN)</th>
<th>Peak Load $2^{nd}$ (kN)</th>
<th>Axial Displacement at $P_u$ $\Delta u$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FTC3</td>
<td>-</td>
<td>610</td>
<td>630</td>
<td>-</td>
<td>-</td>
<td>1.35</td>
</tr>
<tr>
<td>MC3</td>
<td>-</td>
<td>1116</td>
<td>1130</td>
<td>-</td>
<td>-</td>
<td>1.16</td>
</tr>
<tr>
<td>2M0S1</td>
<td>0.41</td>
<td>857</td>
<td>912</td>
<td>912</td>
<td>-</td>
<td>1.32</td>
</tr>
<tr>
<td>2M100S1</td>
<td>0.91</td>
<td>1054</td>
<td>1096</td>
<td>1096</td>
<td>-</td>
<td>1.18</td>
</tr>
<tr>
<td>2M75S1</td>
<td>1.09</td>
<td>993</td>
<td>1025</td>
<td>1025</td>
<td>-</td>
<td>1.14</td>
</tr>
<tr>
<td>2M50S1</td>
<td>1.25</td>
<td>1022</td>
<td>1076</td>
<td>1076</td>
<td>1055</td>
<td>2.67</td>
</tr>
</tbody>
</table>

Note: * Refer to Table 5.2 in Chapter 5 for details.
Among the Group Two specimens, Specimens MC3 and MC4 had higher ultimate load. The results indicate that when the lateral reinforcement ratio was applied at a range between 0.41%-1.82%, the ultimate load was governed by cementitious matrices. The increase in the volumetric ratio, by reducing the spacing of steel hoops, considerably changed the slope of the descending batch of curves. In other words, it significantly improved the ductility. As shown in Fig. 7.11 and Fig. 7.12, for Specimens 2M751 and 2M75S2, the slope of the descending branch of the axial load and displacement curves was less steep than that of Specimens 2M100S1 and 2M100S2. Specimens 2M50S1 and 2M50S2 presented a load plateau and a slightly
less second peak load. It is noted that Specimen 2M0S1 was repaired due to surface imperfection.

7.4 Testing of Group Three Specimens

Mortar FMB5 (61.6 MPa) and FMB6 (63.5 MPa) were applied to the first and the second replicates of this group, respectively. There was an additional category in this group: steel hoops (spaced at 50 mm) and MHSM confined specimens, marked as 0M50S1 and 0M50S2. The failure mode and the axial load and displacement curves are presented in the following section.

7.4.1 Failure Modes of Group Three Specimens

The failure modes of each category are shown in Fig. 7.13. For Specimens 3M75S1, 3M75S2, 3M50S1 and 3M50S2, when reaching the ultimate load, cover spalled considerably. Vertical cracking was dominated in the wire mesh and mortar tube. However, diagonal cracking along a clearly inclined plane appeared in Specimen 3M100S1, as shown in Fig. 7.14. This indicates that after the formation of the plane, lateral wires ruptured, resulting from the relative movement of the matrix along this plane. Again, a clearly diagonal failure plane is attributed to low volumetric ratio of steel confinement (Mander et al. 1988a).

For Specimens 0M50S1 and 0M50S2, it was observed that past the ultimate load the vertical cracks extended and widened quickly. Large pieces of mortar peeled off at a later stage and steel hoops snapped with a loud sound. When the steel hoops fractured, concrete core became unconfined. During the investigation the tested specimens collapsed, as shown in (d) of Fig. 7.14. By contrast, wire mesh tube can constrain steel reinforcement from relaxation and provide additional uniform confinement to the core.
7.4.2 Axial Load and Axial Displacement of Group Three Specimens

The axial load and axial displacement curves of the first and second replicates of the first group are illustrated in Fig. 7.15 and Fig. 7.16, respectively. The test results are presented in Table 7.5 and Table 7.6. It is noted that Specimen 3M0S1 was repaired due to surface imperfection.
Fig. 7.15 Axial load- axial displacement of Group Three Specimens

Table 7.5 Test results of Group Three Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Reinforcement Volumetric load *Ratio (%)</th>
<th>Yield load $P_y$ (kN)</th>
<th>Ultimate load $P_u$ (kN)</th>
<th>Peak Load $1^{st}$ $2^{nd}$ (kN) (kN)</th>
<th>Axial Displacement at $P_u$ $\Delta_u$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FTC5</td>
<td>-</td>
<td>618</td>
<td>636</td>
<td>- -</td>
<td>1.09</td>
</tr>
<tr>
<td>MC5</td>
<td>-</td>
<td>1100</td>
<td>1152</td>
<td>- -</td>
<td>0.97</td>
</tr>
<tr>
<td>3M0S1</td>
<td>0.59</td>
<td>980</td>
<td>1048</td>
<td>1048 -</td>
<td>1.19</td>
</tr>
<tr>
<td>3M100S1</td>
<td>1.25</td>
<td>1073</td>
<td>1152</td>
<td>1152 -</td>
<td>1.39</td>
</tr>
<tr>
<td>3M75S1</td>
<td>1.50</td>
<td>948</td>
<td>1029</td>
<td>1006 1029</td>
<td>3.14</td>
</tr>
<tr>
<td>3M50S1</td>
<td>1.98</td>
<td>1058</td>
<td>1180</td>
<td>1116 1180</td>
<td>3.63</td>
</tr>
<tr>
<td>0M50S1</td>
<td>1.45</td>
<td>933</td>
<td>972</td>
<td>972 -</td>
<td>1.20</td>
</tr>
</tbody>
</table>

Note: * Refer to Table 5.2 in Chapter 5 for details.
As shown in Fig. 7.15 and Fig. 7.16, the specimens with three layers of wire mesh and steel hoops spaced at 50 mm outperformed the rest in terms of the load carrying capacity and ductility. Specimens 3M75S1, 3M75S2, 3M50S1 and 3M50S2 presented a second peak load. Although the second load was slightly higher than the first peak load, a desirable load plateau was clearly presented. Compared to Specimens 3M50S1 and 3M50S2, Specimens 0M50S1 and 0M502 did not present a second peak load. The contrast indicates that wire mesh improved the behaviour of the confined members. Compared to the specimens confined with three layers of wire mesh and a range amount of steel reinforcement, Specimens 3M0S1 and 3M0S2 had...
relatively high ultimate load (due to repair Specimens 3M0S1 had lower ultimate load than Specimen 3M0S2); however, the slope of the axial load and displacement curves of the descending part was rather steeper.

It is also observed that Specimen 3M0S2 and Specimen 3M100S1 had close ultimate load and similar pattern in the axial load and axial displacement curves, as shown in Fig. 7.17. The difference is that for Specimen 3M100S1, a sudden load drop due to hoop fracture appeared at the late stage beyond the peak load; while for Specimen 3M0S2, no sudden load drop appeared and the strength degradation tended further reduced at the late stage beyond the peak load. This figure indicates that specimens with three layers of wire mesh only ($\rho_{3M} = 0.59\%$) can perform comparably to or potentially better than those with low steel volumetric ratio ($\rho_s = 0.72\%$) only.

![Fig. 7.17 Comparison of Specimens 3M0S2 and 3M100S1](image)

7.5 Test results Summary of Plain Concrete Specimens

The axial load and axial displacement curves of Ø150 x 380 mm plain concrete specimens are plotted in Fig. 7.18 and the results of ductility are presented in Table 7.7. The cylindrical concrete controls of the three groups, marked as FTC1-FTC6, had close results in terms of the ultimate load and ductility. The average ultimate load was 635.6 kN.
Fig. 7.18 Axial load-axial displacement of plain concrete specimens

### Table 7.7 Ductility of plain concrete specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield load $P_y$ (kN)</th>
<th>Ultimate load $P_u$ (kN)</th>
<th>$\Delta_y$ (mm)</th>
<th>$\Delta_u$ (mm)</th>
<th>$\Delta_{ss}$ (mm)</th>
<th>$\mu_y$</th>
<th>$\mu_{ss}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>FTC1</td>
<td>644</td>
<td>667</td>
<td>1.21</td>
<td>1.36</td>
<td>1.536</td>
<td>1.12</td>
<td>1.27</td>
</tr>
<tr>
<td>FTC2</td>
<td>606</td>
<td>627</td>
<td>1.23</td>
<td>1.41</td>
<td>1.646</td>
<td>1.14</td>
<td>1.34</td>
</tr>
<tr>
<td>FTC3</td>
<td>610</td>
<td>630</td>
<td>1.20</td>
<td>1.35</td>
<td>1.508</td>
<td>1.12</td>
<td>1.25</td>
</tr>
<tr>
<td>FTC4</td>
<td>602</td>
<td>618</td>
<td>1.20</td>
<td>1.32</td>
<td>1.618</td>
<td>1.11</td>
<td>1.35</td>
</tr>
<tr>
<td>FTC5</td>
<td>618</td>
<td>636</td>
<td>0.98</td>
<td>1.09</td>
<td>1.232</td>
<td>1.12</td>
<td>1.26</td>
</tr>
<tr>
<td>FTC6</td>
<td>604</td>
<td>621</td>
<td>1.20</td>
<td>1.37</td>
<td>1.606</td>
<td>1.15</td>
<td>1.34</td>
</tr>
</tbody>
</table>

### 7.6 Test Results Summary of MHSM Confined Specimens

The test results of MHSM confined specimens, marked as MC1-MC6, are summarised in this section. The axial load and axial displacement curves of Specimens MC1-MC6 are plotted in Fig. 7.19. The ductility ratios are presented in Table 7.8.
As shown in Table 7.8, MHSM confined specimens had relatively high ultimate load. The axial load and displacement curves demonstrated a brittle nature of mortar confined specimens. The bonding agent enhanced MHSM shell and concrete core to work together. As can be seen, MHSM shell did not completely bulged off from the concrete core at the end of test. This was contrast to Specimen TMC. Refer to Section 4.3 for details.

The lateral confinement contributed by mortar is considered negligible in this study (AS3600-2009). At the ultimate load point, load was carried out by cementitious
matrices, including concrete core and MHSM shell. Past the ultimate load, cracks expanded and widened further. The strain gauge readings on the core of MHSM confined specimens consistently showed that the lateral strain of the concrete core increased suddenly and considerably, while the load carrying capacity dropped rapidly for each specimen. This phenomenon was observed approximately ranged from right after the ultimate load to 75% past the ultimate load. Huge jump in the lateral strain readings and rapid strength degradation beyond the peak load indicated the dilation of concrete core and a lack of confinement.

7.7 Test Results of Wire Mesh and MHSM Confined Specimens

7.7.1 Test Results of Experiment III

To facilitate comparing the effect of wire mesh layers (volume fraction of reinforcement, $V_f$), the axial load and axial displacement curves of wire mesh confined specimens are plotted in Fig. 7.20. The ductility results are shown in Table 7.9. It is noted that Specimens 1M0S1, 1M0S2, 2M0S1 and 3M0S1 were repaired due to surface imperfection.

![Fig. 7.20 Axial load- axial displacement of wire mesh confined specimens](image)

Fig. 7.20 Axial load- axial displacement of wire mesh confined specimens
As shown in Fig. 7.20, mesh reinforcement was not sufficient to provide a second peak load. However, the strength degradation tended to develop less rapidly with the increase of reinforcement. The investigation of the tested specimens showed that remained mortar was bonded to the concrete core. Based on the reading of strain gauges, the lateral expansion of the concrete core tended to increase at the ultimate load and increased rapidly after passing the ultimate load. However, the load did not drop rapidly as only MHSM confined specimens. Load was sustained better by applying 3 layers of wire mesh. It is noted that Specimen 2M0S1 had much lower strength than Specimen 2M0S2, which was probably due to the repair over a large area. The results and investigation indicated that wire mesh confinement became effective at a later stage and the contribution of wire mesh could not balance the cover loss.

Compared to only MHSM confined specimens, Table 7.9 shows that: (1) applying one layer of wire mesh did not increase the ultimate load; (2) the specimens confined with two or three layers of wire mesh had similar ultimate load except Specimen 2M0S1; and 3) however, the ductility improved significantly. The results suggest that applying two or three layers of wire mesh can significantly enhance ductility, although the ultimate load did not increase or increased marginally.

### Table 7.9 Ductility of wire mesh confined specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield load $P_y$ (kN)</th>
<th>Ultimate load $P_u$ (kN)</th>
<th>Axial Displacement $\Delta_y$ (mm)</th>
<th>$\Delta_u$ (mm)</th>
<th>$\Delta_{85}$ (mm)</th>
<th>Ductility Ratio $\mu_y$</th>
<th>$\mu_{85}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1M0S1</td>
<td>1036</td>
<td>1086</td>
<td>1.260</td>
<td>1.478</td>
<td>1.858</td>
<td>1.17</td>
<td>1.47</td>
</tr>
<tr>
<td>1M0S2</td>
<td>1014</td>
<td>1064</td>
<td>1.199</td>
<td>1.353</td>
<td>1.673</td>
<td>1.13</td>
<td>1.40</td>
</tr>
<tr>
<td>2M0S1</td>
<td>857</td>
<td>912</td>
<td>1.093</td>
<td>1.319</td>
<td>2.209</td>
<td>1.21</td>
<td>2.02</td>
</tr>
<tr>
<td>2M0S2</td>
<td>1062</td>
<td>1113</td>
<td>1.206</td>
<td>1.467</td>
<td>1.733</td>
<td>1.22</td>
<td>1.44</td>
</tr>
<tr>
<td>3M0S1</td>
<td>980</td>
<td>1048</td>
<td>0.840</td>
<td>1.194</td>
<td>2.646</td>
<td>1.42</td>
<td>3.15</td>
</tr>
<tr>
<td>3M0S2</td>
<td>1101</td>
<td>1153</td>
<td>1.233</td>
<td>1.353</td>
<td>2.744</td>
<td>1.10</td>
<td>2.23</td>
</tr>
</tbody>
</table>
experiments are presented in Fig. 7.21 and Fig. 7.22, respectively. The confined cores were identically 150 mm in diameter for Experiment I and Experiment III. As MSC was cast with wire mesh tube for Experiment II, the confined cores were less 150 mm, varying with the cover thickness and the number of mesh layers. Refer to Table 3.9 in Chapter 3 for the details. It is noted that the cross section areas were not identical for the three series of specimens.
These two figures exhibit the characteristics of HSC, MSC and NSC under compression. The strength degradation rate beyond the peak load and the ultimate vertical deformation were significantly affected by the compressive strength of concrete and reinforcement ratio. Although compared to plain HSC, wire mesh changed the failure mode, two layers of wire mesh was not sufficient to improve ductility significantly. By contrast, two layers were sufficient to improve ductility for MSC specimens. Wire mesh confined NSC demonstrated a gradual strength degradation. It seems that three layers of wire mesh enhanced a better performance.

7.8 Different Layers of Wire Mesh with Same Steel Volumetric Ratio ($\rho_s$)

To investigate how the volume fraction ratio ($V_f$) affects the behavior of the specimens confined with steel hoops, wire mesh and MHSM, the axial load-axial displacement curves of the specimens with the same steel reinforcement ratio ($\rho_s$) but different number of mesh layers (volume fraction, $V_f$) are illustrated in the following sections.

7.8.1 Steel Hoops Spaced at 100 mm ($\rho_s = 0.72 \%$)

As shown in Fig. 7.23, for the specimens confined with steel hoops spaced at 100 mm, the load carrying capacity and ductility improved with the increase in the number of mesh layers. It is noted that each specimens was cast with different batch of mortar, which influenced the load carrying capacity. Specimen 3M100S2 exhibited a rather smooth curve past the ultimate load and before the first steel hoop fracture, evidencing the enhancement in ductility. This figure tends to show that the load at the first steel fracture was improved with the increase of the number of wire mesh layers, which indicated further deformation and energy absorption contributed by wire mesh.
7.8.2 Steel Hoops Spaced at 75 mm ($\rho_s = 0.97 \%$)

As shown in Fig. 7.24, with the increase in the number of mesh layers, the shape of the axial load and displacement curves changed greatly. Compared to Specimens 1M75S1 and 1M75S2, the load dropped more gradually for Specimens 2M75S1 and 2M75S2. Specimens 3M75S1 and 3M75S2 illustrated a desirable load plateau, which indicated that at the same volumetric ratio of steel reinforcement, a slight increase in the volume fraction, $V_f$, from 0.008 to 0.012 can significantly improve ductility. It tends to show that the first steel load increased with the increase of the number of wire mesh layers.
7.8.3 Steel Hoops Spaced at 50 mm ($\rho_s = 1.45\%$)

As shown in Fig. 7.25, all the steel reinforced specimens with a spacing of 50 mm illustrated a rather gradual decline on the descending batch of curves. The load carrying pattern changed significantly with the increment of volume fraction ($V_f$). Specimens 2M50S1 and 2M50S2 presented a load plateau. Specimens 3M50S1 and 3M50S2 illustrated a second peak load. Although part of data was lost, Specimen 1M50S2 presented a load plateau over a long axial displacement.

![Fig. 7.25 Axial load- axial displacement (s = 50 mm)](image)

Compared to Specimens 0M50S1 and 0M50S2, the descending batch of the axial load and displacement curves of Specimens 3M50S1 and 3M50S2 was smooth with significant larger vertical deformation. Fig. 7.25 clearly shows that for Specimens 3M50S1 and 3M50S2 when steel hoop fractured, the load dropped significantly less than their counter parts Specimens 0M50S1 and 0M50S2. With the presence of three layers of wire mesh, the behaviour of confined concrete improved significantly. It is noted that Specimen 0M50S1 was repaired.

Based on Fig. 7.23 - Fig. 7.25, it is concluded that: 1) the shape of the decending batch of the axial load and axial displacement curves is pronouncedly governed by the volumetric ratio of steel reinforcement; 2) as the main reinforcement, the skeleton
of the axial load-axial displacement curves are mainly governed by steel volumetric ratio; and 3) combined with steel reinforcement, the slight increment in volume fraction, $V_f$, can greatly improve ductility at relatively low reinforcement ratio. It shows that the specimens with combinations of $\rho_s = 0.97\%$ with $\rho_w = 0.37\%$, and $\rho_s = 1.45\%$ with $\rho_w = 0.53\%$ had better performance in this case.

7.9  Cover Unloading

As mentioned in Chapter 2, a transaction period occurred where protective cover started unloading (Sheikh and Uzumeri 1980; Sharma et al. 2005; Tavio et al. 2012). Refer to Fig. 2.5 in Chapter 2 for details. For the case of strengthening, the load may be non-dimensionalised with respect to the capacity of concrete core and mortar in the reinforcement tube for the upper curve, while with respect to the capacity of cementitious matrices, including the mortar cover, for the lower curve.

It is not practical to identify the exact point when cover stopped contributing to the load carrying capacity. Two specified strains are recommended for the starting and finishing points of the transaction period, which are corresponding to the maximum plain concrete stress and 50 percent of the peak stress of plain concrete on the descending branch of its stress-strain curve (Sheikh and Uzumeri 1980; Sheikh and Toklucu 1993). In the case of strengthening, the two specified strain values are expected differently. It might be reasonable to use a best-fit curve to find out the transaction period. However, due to the inherent limitations of instrumentation, these two strain values could not be validated in this study.

The cover loses its load gradually while the passive confinement of the core gradually becomes active. Once the cover has spalled off or become loose, the axial load is carried by the confined core and the reinforcement tube. The gain in the strength of the confined core can be overestimated if the peak load occurred at a stage while cover partially sustains the load, but not taken into account.
7.10 Ductility and Energy Absorption of Steel Hoops, Wire Mesh and MHSM Confined Specimens

7.10.1 Displacement Ductility

Different ductility factors were adopted by various researchers in the performance-based design of confining reinforcement. Axial ductility is used to assess column ductility under uniaxial compression (Pessiki and Pieroni 1997; Hadi 2005). Displacement ductility and curvature ductility factors are commonly adopted to determine the flexure ductility of concrete columns (Park and Paulay 1975; Watson and Park 1994).

The axial ductility is recommended as a measure of how much loss of load carrying capacity occurs under increasing axial deformation once the cover region begins to fail. This ductility ratio gives an indication of the amount of energy that can be absorbed before final collapse occurs (Hadi 2005). The curvature ductility factor refers to the section ductility (Sakai and Sheikh 1989) and indicates the capacity of dissipating seismic energy without reduction of the strength (Trezos 1997).

Both axial ductility and curvature ductility factors provide indication about the deformation capacity and energy absorption, and both are influenced by concrete strength, and amount, grade, spacing of transverse reinforcement. The curvature ductility comes from ductility from both the tension side and the compression side, where the compression side is directly related to the axial ductility investigated in the present study. The exact relationship between the two ductility factors depends on many factors, and is beyond the scope of the present study. Further research is needed to establish the relationship between the axial ductility factor and the curvature ductility factor for strengthened columns. In the present study ductility ratio means axial ductility ratio since all the specimens were tested under concentric loading.

To provide an overview on ductility at different stages, three methods were adopted in this study. The first method \( (\mu_y) \) and the second method \( (\mu_{\text{ss}}) \) were as explained in Chapter 3. The third method, ductility at first hoop fracture \( (\mu_s) \) was calculated as \( \mu_s = \mu_{\text{ss}} \).
\[ \Delta u = \Delta u \times \Delta y, \]  where \( \Delta u \) = axial displacement at the first steel hoop fracture. Fig. 7.26 illustrates how each of the displacement was determined.

![Fig. 7.26 Definition of ductility ratios](image)

This figure illustrates a load plateau. Accordingly, the yield load is close to the first peak load. In some case the first steel hoop fractured at 75% past the ultimate load. Due to data loss, the ductility ratio \( \mu_y \) or/and \( \mu_{.85} \) were not available for some of the specimens. Refer to Table 7.10 - Table 7.12 and Fig. 7.25 - Fig. 7.27 for the ductility analysis of the steel hoops, wire mesh and MHSM confined specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield load ( P_y )(kN)</th>
<th>Ultimate load ( P_u )(kN)</th>
<th>Axial Displacement</th>
<th>Ductility Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \Delta y ) (mm)</td>
<td>( \Delta u ) (mm)</td>
<td>( \Delta_{.85} ) (mm)</td>
<td>( \Delta_{sf} ) (mm)</td>
</tr>
<tr>
<td>1M100S1</td>
<td>948</td>
<td>1009</td>
<td>1.184</td>
<td>1.464</td>
</tr>
<tr>
<td>1M100S2</td>
<td>833</td>
<td>914</td>
<td>1.275</td>
<td>1.811</td>
</tr>
<tr>
<td>1M75S1</td>
<td>1039</td>
<td>1100</td>
<td>1.153</td>
<td>1.290</td>
</tr>
<tr>
<td>1M75S2</td>
<td>839</td>
<td>919</td>
<td>1.275</td>
<td>2.045</td>
</tr>
<tr>
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<td>1176</td>
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<td>1.415</td>
</tr>
<tr>
<td>1M50S2</td>
<td>893</td>
<td>958</td>
<td>1.263</td>
<td>1.550</td>
</tr>
</tbody>
</table>

Note:
N1: Due to data loss, the fracture load of this group of specimens was not available. Through investigating the tested specimens, it is confirmed that one steel hoop was broken in Specimens 1M100S1, 1M100S2, 1M75S1; two steel hoops were broken in Specimens 1M75S2, 1M50S1 and 1M50S2.
N2: for Specimen 1M50S2, data lost at 937.7 kN, 98% past the ultimate load.
As shown in Fig. 7.27, for the two replicates of each category, the ductility ratio $\mu_y$ did not change significantly with the increase of steel reinforcement ratio ($\rho_s$). It is noted although the data of Specimen 1M50S2 lost, the load and displacement curve illustrates nearly horizontal plateau up to 5.18 mm and tended to continue, which indicated that its value of $\Delta_{.85}$ is likely to be larger that of Specimen 1M75S2. The second replicates tend to show that with the increase of reinforcement ratio ($\rho_s$), ductility ratio $\mu_{.85}$ increased. This trend was not illustrated by the first replicates, which is probably due to the large difference in the compressive strength of the two batches of mortar.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield load $P_y$ (kN)</th>
<th>Ultimate load $P_u$ (kN)</th>
<th>$\Delta_y$ (mm)</th>
<th>$\Delta_u$ (mm)</th>
<th>$\Delta_{.85}$ (mm)</th>
<th>$\Delta_{sf}$ (mm)</th>
<th>$\mu_y$</th>
<th>$\mu_{.85}$</th>
<th>$\mu_{sf}$</th>
<th>$\Delta_y/\Delta_u$</th>
<th>$\Delta_{.85}/\Delta_u$</th>
<th>$\Delta_{sf}/\Delta_y$</th>
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</thead>
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<tr>
<td>2M100S1</td>
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<td>1096</td>
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<td>7.379</td>
<td>1.09</td>
<td>1.60</td>
<td>6.81</td>
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<td>7.353</td>
<td>1.25</td>
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<td>4.686</td>
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<td>1.05</td>
<td>4.32</td>
<td>6.94</td>
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<td></td>
<td></td>
</tr>
<tr>
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<td>1061</td>
<td>1.333</td>
<td>1.480</td>
<td>4.654</td>
<td>7.855</td>
<td>1.11</td>
<td>3.49</td>
<td>5.89</td>
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<td></td>
<td></td>
</tr>
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<td>1076</td>
<td>1.173</td>
<td>2.672</td>
<td>8.773</td>
<td>8.846</td>
<td>2.28</td>
<td>7.48</td>
<td>7.54</td>
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<td></td>
<td></td>
</tr>
<tr>
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<td>992</td>
<td>1034</td>
<td>1.102</td>
<td>1.491</td>
<td>7.206</td>
<td>7.841</td>
<td>1.35</td>
<td>6.54</td>
<td>7.11</td>
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<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 7.11 Ductility ratios Group Two Specimens
Table 7.11 and Fig. 7.28 show that with the presence of two layers of wire mesh, the ductility ratio $\mu_y$ increased slightly with the decrease in the hoop spacing, while the ductility ratio $\mu_{85}$ increased significantly. In addition, the ductility ratio $\mu_{sf}$ had significant large value and tended to increase with the decrease in hoop spacing, especially for Specimens 2M50S1 and 2M50S2. The value of $\mu_{85}$ is a measure of the ductility at the early stage of the post peak load. The larger value of $\mu_{85}$ shows the larger deformation and indicates more ductile behaviour. For the two specimens of each category, load at the first fracture occurred at a similar proportion of the ultimate load. It is noted that although the specimens of each category had a similar axial deformation at the first steel fitment fracture, the specimens with less value in $\Delta_y$ had larger value in $\mu_{sf}$. With the decrease in the spacing of steel hoops (increase in $\rho_s$), load at the first steel hoop fracture was clearly increased, indicating that steel enforcement became activated at an earlier stage.
Table 7.12 Ductility ratios of Group Three Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield load $P_y$ (kN)</th>
<th>Ultimate load $P_u$ (kN)</th>
<th>Axial Displacement $\Delta_y$ (mm)</th>
<th>Ultimate Axial Displacement $\Delta_u$ (mm)</th>
<th>Ductility Ratio $\Delta_{u/\Delta_y}$</th>
<th>$\Delta_{u/\Delta_y}^{\mu_y}$</th>
<th>$\Delta_{u/\Delta_y}^{\mu_{85}}$</th>
<th>$\Delta_{u/\Delta_y}^{\mu_{sf}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3M100S1</td>
<td>1073</td>
<td>1152</td>
<td>0.918</td>
<td>1.388</td>
<td>3.040</td>
<td>5.613</td>
<td>1.51</td>
<td>3.31</td>
</tr>
<tr>
<td>3M100S2</td>
<td>942</td>
<td>1038</td>
<td>1.337</td>
<td>2.433</td>
<td>6.240</td>
<td>7.401</td>
<td>1.82</td>
<td>4.67</td>
</tr>
<tr>
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<td>948</td>
<td>1029</td>
<td>1.013</td>
<td>3.140</td>
<td>5.890</td>
<td>6.487</td>
<td>3.10</td>
<td>5.82</td>
</tr>
<tr>
<td>3M75S2</td>
<td>941</td>
<td>1031</td>
<td>1.265</td>
<td>4.782</td>
<td>*</td>
<td>*</td>
<td>3.78</td>
<td>*</td>
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<tr>
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<td>1.107</td>
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<td>6.821</td>
<td>3.28</td>
<td>5.90</td>
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<td>3.726</td>
<td>4.441</td>
<td>1.42</td>
<td>4.41</td>
</tr>
<tr>
<td>0M50S2</td>
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<td>1093</td>
<td>1.257</td>
<td>1.364</td>
<td>4.650</td>
<td>6.526</td>
<td>1.08</td>
<td>3.70</td>
</tr>
</tbody>
</table>

Note:
*Specimen 3M75S2: data was recorded up to 957 kN, 93% past the ultimate load, at which point the axial displacement reached 6.31 mm.

Fig. 7.29 Ductility of Group Three Specimens

Table 7.12 and Fig. 7.29 indicate that the first replicates (Specimens 3M100S1, 3M75S1 and 3M50S1) show that with the presence of three layers of wire mesh, the three ductility ratios ($\mu_y$, $\mu_{85}$, $\mu_{sf}$) increased with the decrease in spacing. A similar pattern can be expected for the second replicates. Although the full data of Specimen 3M75S1 was not available, Specimen 3M75S2 demonstrated a larger load plateau than Specimen 3M75S1, indicating a likely higher value in ductility. Specimens 3M75S1,2 and 3M50S1,2 exhibited a large increment in displacement from $\Delta_y$ to $\Delta_u$, which reflected a load plateau or a second peak load. However, compared to
Specimens 3M75S1 and 3M75S2, the increment in ductility ratios of Specimens 3M50S1 and 3M50S2 was not significant. Again, with the decrease in hoop spacing, the first steel hoop fracture occurred closer to the ultimate load. It is noted that for Specimen 3M50S2, the first steel fracture occurred at 90% past the ultimate load. The value of $\mu_{85}$ was obtained by linear interpolation. The ductility ratios of the specimens confined with steel hoops and three layers of wire mesh significantly outperformed Specimens 0M50S1 and 0M50S2.

It is noted that comparing ductility needs to consider the axial load and axial displacement curve. For instance, compared to Specimen 0M50S1, Specimen 0M50S2 had considerably large values in $\Delta_y$, $\Delta_{85}$ and $\Delta_{sf}$. However, its $\mu_{85}$, $\mu_{sf}$ values were smaller than that of Specimen 0M50S1, despite the fact that Specimen 0M50S2 outperformed Specimen 0M50S1 in terms of load carrying capacity and ductility. Similar situation occurred to Specimens 3M100S2, 3M50S2.

7.10.2 First Steel Hoop Fracture

According to Scott et al. (1982), the axial strain at the point of the first hoop fracture occurs is considered as the ultimate strain, which was not able to obtain in this experiment due to equipment constraints. However, the investigation was carried on the load and the axial displacement when the first steel hoop fracture occurred, as shown in Table 7.13.
Table 7.13 First steel hoop fracture

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield load $P_y$ (kN)</th>
<th>Ultimate load $P_u$ (kN)</th>
<th>First Steel Hoop Fracture</th>
<th>Compared to $P_u$ (%)</th>
<th>Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1M100S1</td>
<td>948</td>
<td>1009</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>1M100S2</td>
<td>833</td>
<td>914</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>1M75S1</td>
<td>1039</td>
<td>1100</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>1M75S2</td>
<td>839</td>
<td>919</td>
<td>*</td>
<td>*</td>
<td>*</td>
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<tr>
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<td>1176</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>1M50S2</td>
<td>893</td>
<td>958</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>2M100S1</td>
<td>1102</td>
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</tr>
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<td>590</td>
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<td>*</td>
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<td>1101</td>
<td>990</td>
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<td>1093</td>
<td>835</td>
<td>76.4</td>
<td>6.526</td>
</tr>
</tbody>
</table>

Note: *Due to data loss, the fracture load of this group of specimens was unable to be provided. Through investigating the tested specimens, it is confirm that One steel hoop was broken in Specimens 1M100S1, 1M100S2 and 1M75S1; two broken in Specimens 1M75S2, 1M50S1 and 1M50S2; 3M75S2: due to the loss of data, 957k N, 93% past the ultimate load, at which point the axial displacement reached 6.31 mm.

Table 7.13 indicates that: (1) for the same number of wire layers ($V_f$), with the decrease in hoop spacing, the first steel hoop fractured closer to the ultimate load; (2) for the same volumetric ratio of lateral steel ($\rho_s$), with the increase of the number of mesh layers, the first steel fracture tended to occur sooner past the ultimate load.

7.10.3 Energy Absorption

As mentioned in Section 3.7 of Chapter 3, the areas under the axial load and axial displacement curves (AUC), indicates the energy absorption capacity and the ductility of the specimen (Ganesan and Anil 1993). As the testing for all the
specimens did not finish at the same post ultimate load, 75% past the ultimate load was set as the boundary of AUC for each specimen of Group One, while 50% past the ultimate load was set for Group Two and Three. Accordingly, two factors ($I_{75}$ and $I_{50}$) were adopted as energy absorption index. Refer to Section 3.7 in Chapter 3 for the explanation of $I_{50}$. The index $I_{75}$ was adopted for this group following the same analogy as for $I_{50}$. The values of the energy absorption index are summarised in Table 7.14-Table 7.16.

### Table 7.14 Energy absorption Index, $I_{75}$, Group One

<table>
<thead>
<tr>
<th>Specimen</th>
<th>AUC (kN.mm)</th>
<th>Proportion of $P_u$ (%)</th>
<th>Energy absorption Index, $I_{75}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>MC1</td>
<td>1062</td>
<td>-</td>
<td>1</td>
</tr>
<tr>
<td>MC2</td>
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<td>-</td>
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</tr>
<tr>
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</tr>
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<td>4.18</td>
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### Table 7.15 Energy absorption Index, $I_{50}$, Group Two

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<th>Specimen</th>
<th>AUC (kN.mm)</th>
<th>Proportion of $P_u$ (%)</th>
<th>Energy absorption Index, $I_{50}$</th>
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</thead>
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<td>MC4</td>
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<td>-</td>
<td></td>
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<tr>
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<td>4.04</td>
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Table 7.16 Energy absorption Index, $I_{50}$, Group Three

<table>
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<th>Proportion of $P_e$ (%)</th>
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<td>4.2</td>
</tr>
<tr>
<td>3M100S2</td>
<td>6851</td>
<td>53</td>
<td>5.9</td>
</tr>
<tr>
<td>3M75S1</td>
<td>6342</td>
<td>50</td>
<td>5.4</td>
</tr>
<tr>
<td>3M75S2</td>
<td>5723</td>
<td>93</td>
<td>4.9</td>
</tr>
<tr>
<td>3M50S1</td>
<td>9246</td>
<td>50</td>
<td>7.9</td>
</tr>
<tr>
<td>3M50S2</td>
<td>9600</td>
<td>50</td>
<td>8.2</td>
</tr>
<tr>
<td>0M50S1</td>
<td>3673</td>
<td>50</td>
<td>3.2</td>
</tr>
<tr>
<td>0M50S2</td>
<td>6722</td>
<td>50</td>
<td>5.8</td>
</tr>
</tbody>
</table>

Table 7.14 - Table 7.16 show that steel hoops absorb significant energy; the AUC value increased considerably with the significant increase of the lateral steel reinforcement volume ratio. For instance, reducing hoop spacing from 100 mm to 75 mm (equivalently improving reinforcement ratio from 0.72% to 0.97%) did not affect energy absorption significantly, while reducing hoop spacing from 75 mm to 50 mm (equivalently improving reinforcement ratio from 0.97% to 1.45%) did affect energy absorption significantly; 3 layers of wire mesh absorbed significant energy. In general, energy absorption index increases with the increase of $V_f$ and $\rho_s$. Steel hoops with relatively high reinforcement ratio outperformed wire mesh. However, two and three layers of wire mesh made comparable contribution, compared with specimens confined with low steel reinforcement ratio.

7.11 Strains of Steel Hoops

Investigation of the strains on steel hoops was conducted. As shown in Table 7.17, when it was 90% prior to the ultimate load, the lateral strain in steel ties was marginal for all the specimens. At this point the confinement caused by transverse reinforcement was negligible. At the ultimate load the strain in the steel hoops increased to varying degrees among all the groups, depending on the reinforcement
volumetric ratio. The increment was considerable for the group reinforced with three layers of wire mesh.

Among the specimens of Group Three, at the ultimate load, the strain gauge reading of Specimens 3M50S1 and 3M50S2 is considerably increased, indicating significant dilation in the concrete core and activation of the passive confining pressure of the hoop and wire mesh. By contrast, the lateral strain reading in the core of Specimens 0M50S1 and 0M50S2 was small. The strain gauge readings and the axial load and axial displacement curves suggest that for most of the specimens, at the ultimate load point, the load was mainly contributed by concrete and mortar, except for Specimens 3M75S1, 3M75S2, 3M50S1 and 3M50S2. For these four specimens, steel confinement became effective, but not fully active, at the ultimate load or shortly past the ultimate load. For Specimens 2M50S1 and 2M50S2, the strains of steel reinforcement were significant around the second peak load. For Specimen 1M50S2 the strain gauges were damaged after the first peak load. However, the load was sustained almost as the first peak load over a large displacement and by reaching the second peak the cover nearly completely peeled off. Therefore, it is reasonable to consider the steel confinement became activated at the second peak load. As the main confinement, the characteristics of steel confinement have significant influence on the behaviour of the confined members.
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Average hoop strain at 0.9 $P_u$ ($\mu e$)</th>
<th>Average hoop strain at $P_u$ ($\mu e$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1M100S1</td>
<td>75</td>
<td>843</td>
</tr>
<tr>
<td>1M100S2</td>
<td>94</td>
<td>574</td>
</tr>
<tr>
<td>1M75S1</td>
<td>332</td>
<td>398</td>
</tr>
<tr>
<td>1M75S2</td>
<td>-</td>
<td>(1)</td>
</tr>
<tr>
<td>1M50S1</td>
<td>262</td>
<td>406</td>
</tr>
<tr>
<td>1M50S2</td>
<td>446</td>
<td>(2)</td>
</tr>
<tr>
<td>2M100S1</td>
<td>87</td>
<td>133</td>
</tr>
<tr>
<td>2M100S2</td>
<td>134</td>
<td>178</td>
</tr>
<tr>
<td>2M75S1</td>
<td>133</td>
<td>143</td>
</tr>
<tr>
<td>2M75S2</td>
<td>74</td>
<td>155</td>
</tr>
<tr>
<td>2M50S1</td>
<td>117</td>
<td>664</td>
</tr>
<tr>
<td>2M50S2</td>
<td>144</td>
<td>667</td>
</tr>
<tr>
<td>3M100S1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3M100S2</td>
<td>151</td>
<td>1587</td>
</tr>
<tr>
<td>3M75S1</td>
<td>351</td>
<td>2365</td>
</tr>
<tr>
<td>3M75S2</td>
<td>210</td>
<td>3781</td>
</tr>
<tr>
<td>3M50S1</td>
<td>345</td>
<td>1688</td>
</tr>
<tr>
<td>3M50S2</td>
<td>296</td>
<td>3219</td>
</tr>
<tr>
<td>STL50S1</td>
<td>489</td>
<td>1043</td>
</tr>
<tr>
<td>STL50S2</td>
<td>451</td>
<td>708</td>
</tr>
</tbody>
</table>

**Note:**
- Strain gauges damaged before testing
(1) strain gauges on steel hoop damaged by wire mesh due to concrete core dilation
(2) strain gauges became unavailable shortly past the peak load

According to Ahmad and Shah (1982) and Li et al. (2001), passive confinement is less efficient for ultrahigh strength steel confined concrete. Ahmad and Shah (1982) indicated that for high strength steel reinforcement, the lateral strain of concrete increased rapidly beyond the peak of stress-strain curve and caused the stress in spiral also increased rapidly. Li et al. (2001) pointed that there is a delayed confining effect on the concrete relative to that provided by normal yield strength steel because the ultra-high yield strength of steel is not developed until at high transverse strains. Furthermore, Razvi and Saatcioglu (1999) reported that transverse reinforcement remains within the elastic range when HSC is confined with high-strength reinforcement. Additionally, in studies on high strength spiral reinforced compression members, Pessiki et al (2001) observed that the failures were not due to
the spiral reinforcement reaching its ultimate strain as caused by lateral expansion of concrete core.

The steel adopted in this study was relatively high, having a yield stress of 658 MPa, as shown in Table 6.5 of Chapter 6. This yield stress was in complying with the specification in AS3600 (2009) and ACI 318 (2008). The steel fitment yield stress is specified as not greater than 800 MPa and 700 MPa, respectively, in AS3600 (2009) and ACI 318 (2008). The yield stress of transverse steel reinforcement is generally used for determining the confining pressure. This study confirms that steel hoops with relatively high strength did not reach the yield stress at the second peak load. Therefore, if the yield strength of lateral reinforcement is adopted for calculating the confining pressure, it could lead to overestimation. It is noted that NSC was used for casting the core for the third series of experiments.

Due to the confinement constraints, some of the strain gauges on the steel hoops failed or damaged at an early stage. However, steel hoops were exhibited significant dilation. For specimens confined with one layer of wire mesh, the measured average extension of the fractured steel hoop was 1.79%, 1.82% and 3.09% for Specimens 1M100S1,2, Specimens1M75S1,2, and Specimens 1M50S1,2, respectively; while for Specimens 2M100S1,2, Specimens2M75S1,2, and Specimens 2M50S1,2, it was 2.86%, 5.15% and 5.21%, respectively.

7.12 Comparison of Experimental Results and Estimated Values

The use of lateral confinement to improve the strength and ductility of concrete members in compression has been a constant research interest. To investigate the effect of confinement, the experimental results of the ultimate load are compared with the estimations based on different models, as indicated in Chapter 2. The confined specimens are classified into three types: 1) MHSM confined specimens, 2) wire mesh and MHSM confined specimens, 3) steel hoops, wire mesh and MHSM confined specimens.
7.12.1 Ultimate Load Estimation for Mortar Confined Specimens

The bonding between the core and mortar shell was enhanced by polymer agent EMACO157. According to Jiang et al. (2009), the deformation of mortar shell and concrete core was considered uniformly up to 85% of the ultimate load. The average in-suit compressive strength of Ø100 × 200 mm cylinders and Ø150×380 mm cylindrical specimens was 40 MPa and 35.8 MPa, respectively. The latter was approximately 90% of the former. The strength of Ø150×380 mm specimens (35.8 MPa) was adopted as the strength of the core. The ultimate load was estimated using Eq. 7.1 and presented in Table 7.18. The estimations were reasonably close to the experimental results.

\[ P_t = f'_{cm} A_c + \alpha_{1,m} f'_m \left(D^2 - D_c^2\right)/4 \]  

(7.1)

where \( A_c \) = area of concrete core; \( f_{co} \) = compressive strength of plain concrete (refer to Section 7.5); \( f'_m \) = the compressive strength of mortar; \( D \) = diameter of the specimen; \( D_c \) = diameter of the core; \( \alpha_{1,m}=1.0-0.003 f'_m \), as explained in Chapter 3; \( f'_m \) = compressive strength of mortar at 28 days (refer to Chapter 6 for details).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( f'_m ) (MPa)</th>
<th>Tested Load ( P_{exp} ) (kN)</th>
<th>Predicted Load ( P_t ) (kN)</th>
<th>Deviation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MC1</td>
<td>62.1</td>
<td>1184</td>
<td>1172</td>
<td>1.0</td>
</tr>
<tr>
<td>MC2</td>
<td>48.5</td>
<td>1194</td>
<td>1075</td>
<td>-9.9</td>
</tr>
<tr>
<td>MC3</td>
<td>59.5</td>
<td>1130</td>
<td>1154</td>
<td>2.2</td>
</tr>
<tr>
<td>MC4</td>
<td>55.6</td>
<td>1088</td>
<td>1127</td>
<td>3.7</td>
</tr>
<tr>
<td>MC5</td>
<td>61.6</td>
<td>1152</td>
<td>1169</td>
<td>1.5</td>
</tr>
<tr>
<td>MC6</td>
<td>63.7</td>
<td>1180</td>
<td>1183</td>
<td>0.3</td>
</tr>
<tr>
<td>AAE (%)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>3.1</td>
</tr>
</tbody>
</table>

7.12.2 Wire Mesh and MHSM Confined Specimens

7.12.2.1 Ultimate Load Estimation

The geometry and parameters of each wire mesh and MHSM confined specimens are shown in Fig. 7.30 and Table 7.19. \( D_c \), \( D_m \) and \( D_w \) are, respectively, the diameter of
the concrete core, the overall dimension of the mean of the wire mesh layers and the
overall dimension measured between the outermost wires. $D_m$ equals to $(D_w + D_c)/2$.

Fig. 7.30 Cross section for wire mesh and MHSM confined specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$D_m$ (mm)</th>
<th>$D_w$ (mm)</th>
<th>Cover $t$ (mm)</th>
<th>No. of Mesh Layers</th>
<th>Reinforcement Ratio $p_w$ (%)</th>
<th>Volume Fraction $V_f$</th>
<th>Ultimate Load $P_u$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1M0S1</td>
<td>153</td>
<td>155</td>
<td>18</td>
<td>1</td>
<td>0.21</td>
<td>0.008</td>
<td>1086</td>
</tr>
<tr>
<td>1M0S2</td>
<td>153</td>
<td>155</td>
<td>18</td>
<td>1</td>
<td>0.21</td>
<td>0.008</td>
<td>1064</td>
</tr>
<tr>
<td>2M0S1</td>
<td>155</td>
<td>159</td>
<td>15</td>
<td>2</td>
<td>0.41</td>
<td>0.016</td>
<td>912</td>
</tr>
<tr>
<td>2M0S2</td>
<td>155</td>
<td>159</td>
<td>15</td>
<td>2</td>
<td>0.41</td>
<td>0.016</td>
<td>1113</td>
</tr>
<tr>
<td>3M0S1</td>
<td>158</td>
<td>166</td>
<td>12</td>
<td>3</td>
<td>0.59</td>
<td>0.024</td>
<td>1048</td>
</tr>
<tr>
<td>3M0S2</td>
<td>158</td>
<td>166</td>
<td>12</td>
<td>3</td>
<td>0.59</td>
<td>0.024</td>
<td>1153</td>
</tr>
</tbody>
</table>

*Note: $p_w$ is respect with the core; $V_f$, is respect with the mortar shell, including vertical wires.

As shown in Table 7.19, the ultimate load fluctuated among the six specimens. It shows that $V_f$ (ranged from 0.008 to 0.024) or $p_w$ (ranged from 0.21% to 0.59%) did not affect the ultimate load significantly. Specimens with different $V_f$ had close value in the ultimate load, except Specimen 2M0S1. Based on the investigation of the tested specimens, this problem was most likely caused by imperfection during the mortar casting process. Workmanship can be an important factor, as extra care is required to ensure mortar to pack through mesh.

On the other hand, the volume fraction ($V_f$) has significant influence on ductility. The specimens with less $V_f$ demonstrated more brittle behaviour while specimens with relatively larger $V_f$ presented a more ductile behaviour, which can be seen from
the descending part of the axial load and axial displacement curves. With the increase of volume fraction $V_f$ (the lateral wire volumetric ratio also increased), the cover spalled more, but in a gradual manner. Furthermore, with the increase in the specific surface of reinforcement ($S_r$), the spacing of vertical cracks reduced. Refer to Fig. 7.3 for details. The ultimate load, the volume fraction and the lateral wire volumetric ratio are plotted in Fig. 7.31.

![Fig. 7.31 Ultimate load and wire mesh reinforcement ratio](image)

A series of confinement models were adopted for estimating the ultimate load of wire mesh and MHSM confined specimens, as presented in Chapter 2. It is noted that the mortar area was from the core to the overall dimension of the mean of the wire mesh layers, $D_m$. For the approach proposed by Xiong et al. (2011), both transvers and vertical wires were taken into account for $\mu_w$. The difference between the estimated and experiment results was reduced for Specimens 3M0S1 and 3M0S2.

In addition, as ferrocement is considered as homogenous and isotropic material, demonstrating rigid and plastic behaviour (ACI 549R-1997), the confining pressure can be determined based on the experiment results of the direct tension test of wire mesh encapsulated mortar plates, using Eq. 7.2:
\[
\ell_{mp}' = \frac{2T}{D_c b}
\]  
(7.2)

where \( \ell_{mp}' \) is the confining pressure based on mesh encapsulated mortar plate; \( T \) is the test results of the ultimate tension load as indicated in Section 6.3 (Chapter 6); \( b \) is the width of mortar plates; \( D_c \) = diameter of concrete core.

For most of the models, the estimated load was significantly less than the experiment result, except the methods proposed by Waliuddin and Rafeeqi (1994), Kaushik and Singh (1997), Xiong et al. (2011) for the specimens confined with two and three layers of wire mesh. At the ultimate load wire mesh confinement was not effective; the ultimate load was contributed mainly by cementitious matrices. It is noted that the compressive strength of mortar was not considered in most of the models. For low strength mortar, for instance, grout with compressive strength of 15 MPa (AS3700), it does not affect the ultimate load as the cover loss can be balanced by wire mesh confinement. However, when applied with high strength matrix, the contribution of wire mesh might not balance the load associated with cover loss. In this case, considering the load is only carried by the confined concrete core at the ultimate load will lead to overestimate the strength of the confined core.

From Experiment II and III, it was observed that increasing the number of mesh layers did not necessarily ensure the increase in load carrying capacity. To the contrary, the ultimate load might slightly reduce, which was also found in other studies (Cheng 2010; Xiong et al. 2011; Ho et al 2013). This is likely because increasing the number of mesh layers equivalently reduced the mortar area.

For the case of applying one to three layers of mesh, the contribution of vertical wires can be considered negligible due to the smallness of wire diameter. Therefore, Eq. 7.1 can also be adopted for estimating the ultimate load \( (P_i) \) for wire mesh and MHSM confined specimens, which provides a reasonable agreement with the experiment results, except for Specimen 2M0S1 as explained earlier. Refer to Table 7.20 for details.
Table 7.20 Wire mesh confined specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Tested Load $P_{\text{exp}}$ (kN)</th>
<th>Predicted Load</th>
<th>Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Eq. 7.1 $P_1$ (kN)</td>
<td>Eq. 7.3 $P_2$ (kN)</td>
<td>$P_1$ (%)</td>
</tr>
<tr>
<td>1M0S1</td>
<td>1086</td>
<td>1172</td>
<td>706</td>
</tr>
<tr>
<td>1M0S2</td>
<td>1064</td>
<td>1075</td>
<td>700</td>
</tr>
<tr>
<td>2M0S1*</td>
<td>912</td>
<td>1154</td>
<td>776</td>
</tr>
<tr>
<td>2M0S2</td>
<td>1113</td>
<td>1127</td>
<td>773</td>
</tr>
<tr>
<td>3M0S1</td>
<td>1048</td>
<td>1169</td>
<td>854</td>
</tr>
<tr>
<td>3M0S2</td>
<td>1153</td>
<td>1183</td>
<td>857</td>
</tr>
<tr>
<td>AAE (%)</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Note: *Specimen 2M0S1 repaired; Deviation = $(P_{\text{mod}} - P_{\text{exp}})/P_{\text{exp}}$

For the case that the transvers confinement of wire mesh is efficient to balance the loss of cover, the ultimate load carried out by the confined concrete core and mortar infill within the reinforcement tube can be estimated using Eq.7.3-Eq.7.5. The confinement effectiveness coefficient, $k_1 = 4.1$ (Richart et al. 1928). Due to the arching action, an effectiveness factor needs to be taken into account to obtain the equivalent lateral confining pressure, which is assumed to be distributed uniformly over the core (Mandar et al. 1989b, Sheikh and Uzumeri 1980, Sheikh and Uzumeri 1982). Based on AS3600 (2009), $k_{ew}$ ranged between 0.92 and 0.93, depending on $D_m$.

$$P_2 = \left( f_{w} + 4.1 k_1 f_{w}' \right) A_e + \alpha_{1,m} f_{w}' \pi / 4 \left( D_m^2 - D_e^2 \right) \quad (7.3)$$

$$f_{w}' = \frac{n^2 A_w f_{sw}}{D_w s_w} \quad (7.4)$$

$$k_{ew} = \left( 1 - \frac{s_w}{2 D_m} \right)^2 \quad (7.5)$$

where $f_{w}'$ = confining pressure due to wire mesh; $s_w$ = opening of square wire mesh; $A_w$ = area of single wire; $f_{co}$, $A_c$, $a_{1,m}$, $f_{w}'$, $n$, $D_m$, and $D_e$ are as defined previously.
For the method of mortar plate, the confining pressure \( f'_{mp} \) is determined using Eq. 7.2, as mentioned earlier; while the ultimate load is calculated using Eq. 7.6. It is noted that no effectiveness factor \( k_e \) is applied for Eq. 7.6.

The load carried by mortar in the reinforcement tube increased with the increase of mortar area. The estimated values based on the two methods were close and significantly less than the corresponding experiment results. In the case of lack experimental data for \( T \), it can be estimated either using Eq. 6.3 or multiplying the average ultimate load of mesh coupon with the corresponding number of mesh layers. Refer to Chapter 6 Section 6.3 for details. The strength of confined core based on Eq. 7.2 was marginally larger than its counterparts calculated using Eq. 7.4 and Eq. 7.5 since no effectiveness factor \( k_e \) was considered.

In addition to the statistic indicator AAE, another indicator - mean square error (MSE) was adopted to evaluate the prediction for the ultimate load. MSE is determined by Eq. 7.7.

\[
MSE = \frac{\sum_{i=1}^{n} \left( \frac{\text{mod}_i - \text{exp}_i}{\text{exp}_i} \right)^2}{n}
\]  

where mod = predicted results, exp = experimental result, and \( n \) is the total number of tested wire mesh reinforced specimens. The values of MSE and AAE for the models with relatively less variation are presented in Table 7.21 and Fig. 7.32.
Table 7.21 Evaluation of estimated ultimate load for wire mesh reinforced specimens

<table>
<thead>
<tr>
<th>Model</th>
<th>Expression</th>
<th>MSE (%)</th>
<th>AAE (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Waliuddin and Rafeeqi 1994</td>
<td>$P_2 = A_c \left( f_{co} + 35\rho k_{w} k_r f_{m} \right)$</td>
<td>8.2</td>
<td>27.7</td>
</tr>
<tr>
<td>Eq. 7.3 (Mortar Infill)</td>
<td>$P_2 = A \left( f_{co} + k_{ew} f'<em>{w} \right) + \alpha</em>{1, w} f_{m} \pi \left( D_{w}^2 - D_{c}^2 \right) / 4$</td>
<td>7.2</td>
<td>25.8</td>
</tr>
<tr>
<td>Eq. 7.6 (Mortar Plate)</td>
<td>$P_2 = A \left( f_{co} + k_{c} f'<em>{mp} \right) + \alpha</em>{1, m} f_{m} \pi \left( D_{w}^2 - D_{c}^2 \right) / 4$</td>
<td>6.9</td>
<td>24.8</td>
</tr>
<tr>
<td>Kaish et. al 2015</td>
<td>$P_2 = A_c \left( f_{co} + 11.2 k_{ew} f'_{w} \right)$</td>
<td>5.0</td>
<td>20.3</td>
</tr>
<tr>
<td>Xiong et al. 2011</td>
<td>$P^r = A_c f_{co} \left( 1 + 1.98 \lambda_{c} \right)$</td>
<td>3.2</td>
<td>13.9</td>
</tr>
<tr>
<td>Rechart et al. 1928</td>
<td>$P_2 = A \left( f_{co} + 4.1 k_{ew} f'_{w} \right)$</td>
<td>11.1</td>
<td>32.9</td>
</tr>
<tr>
<td>Balmer 1949</td>
<td>$P_2 = A_c f_{co} \left( 1 + 9.175 \left( k_{ew} f'<em>{w} \right) / f</em>{co} \right)^{0.73}$</td>
<td>8.2</td>
<td>27.7</td>
</tr>
<tr>
<td>Fafitis and Shah 1985</td>
<td>$P_2 = A \left( f_{co} + \left( 1.15 + \frac{21}{f_{co}} \right) k_{ew} f'_{w} \right)$</td>
<td>13.8</td>
<td>36.8</td>
</tr>
<tr>
<td>Mander et al. 1988a</td>
<td>$P_2 = A \left( 2.254 f_{co} \sqrt{1 + 7.94 \left( k_{ew} f'<em>{w} \right) / f</em>{co} - 2 \left( k_{ew} f'<em>{w} \right) - 1.254 f</em>{co}} \right)$</td>
<td>8.2</td>
<td>27.8</td>
</tr>
<tr>
<td>Samaan et al. 1998</td>
<td>$P_2 = A \left( f_{co} + 6 \left( k_{ew} f'_{w} \right)^{0.7} \right)$</td>
<td>8.9</td>
<td>29.2</td>
</tr>
<tr>
<td>Saatcioglu and Razvi 1999</td>
<td>$P_2 = A \left( f_{co} + 6.7 \left( k_{ew} f'_{w} \right)^{0.83} \right)$</td>
<td>8.1</td>
<td>27.7</td>
</tr>
<tr>
<td>Eq. 7.1 ($P_1$)</td>
<td>$P_2 = A_c f_{co} + \alpha_{1, w} f_{m} \pi \left( D_{c}^2 - D_{c}^2 \right) / 4$</td>
<td>1.5</td>
<td>8.5</td>
</tr>
</tbody>
</table>
7.12.2.2 Cover Thickness

The thickness of wire mesh and mortar composite shell was 20 mm. The load contributed by the protective cover is set not greater than the gain due to wire mesh, shown as Eq. 7.8. For the convenience and simplicity, the confinement effectiveness coefficient $k_1$ was replaced by 4. Based on the measurement in Experiment III, one layer of wire mesh can be estimated approximately as 2.5 mm thick. This value varies as it is depending on the type of mesh, the strengthening method and the number of mesh layers. Therefore, Eq. 7.8 can be rearranged as Eq. 7.9.

$$k_1 \left( \frac{k_{c}}{k_{c,mesh}} \frac{2nA_{mesh}f_{c,mesh}}{D_{m}s_{w}} \right) A \geq \alpha_{t,w}f_{m}^{'}\pi \left( (D_{c} + 5n + 2t)^{2} - (D_{c} + 5n)^{2} \right) / 4$$ (7.8)

$$t^{2} + (D_{c} + 5n)t - \frac{k_{c,mesh}2n\alpha_{t,w}f_{m}^{'}A_{t}}{\alpha_{t,w}f_{m}D_{m}s_{w}} \leq 0$$ (7.9)
where \( k_1 \approx 4; \) \( t = \) thickness of mortar protective cover; \( d_w = \) diameter of wire; \( f_{yw} = \) yield strength of wire; \( s_w = \) centre to centre spacing of wires along the column; \( n = \) number of mesh layers, \( D_m, D_c, A_c, f'_m \) and \( k_{ew} \) are as defined previously. The calculated thickness for each wire mesh confined specimen was shown in Table 7.22.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Calculated cover (mm)</th>
<th>Actual cover (mm)</th>
<th>Mould for Cover</th>
<th>Mortar strength ( f'_m ) (kN)</th>
<th>Ultimate load, ( P_u ) or ( P_2 )</th>
<th>Plateau</th>
</tr>
</thead>
<tbody>
<tr>
<td>1M0S1</td>
<td>1.66</td>
<td>17.7</td>
<td>Y</td>
<td>62.1</td>
<td>1086</td>
<td>N</td>
</tr>
<tr>
<td>1M0S2</td>
<td>2.13</td>
<td>17.4</td>
<td>Y</td>
<td>48.5</td>
<td>1064</td>
<td>N</td>
</tr>
<tr>
<td>2M0S1</td>
<td>3.27</td>
<td>16.0</td>
<td>Y</td>
<td>59.5</td>
<td>912</td>
<td>N</td>
</tr>
<tr>
<td>2M0S2</td>
<td>3.45</td>
<td>14.8</td>
<td>Y</td>
<td>55.6</td>
<td>1113</td>
<td>N</td>
</tr>
<tr>
<td>3M0S1</td>
<td>4.46</td>
<td>12.3</td>
<td>Y</td>
<td>61.6</td>
<td>1048</td>
<td>N</td>
</tr>
<tr>
<td>3M0S2</td>
<td>4.32</td>
<td>12.0</td>
<td>Y</td>
<td>63.5</td>
<td>1153</td>
<td>N</td>
</tr>
</tbody>
</table>

Table 7.22 shows that the actual protective covers were significantly thicker than the theoretical ones, which indicates that the load carried by the cover exceeded the wire mesh contribution. Based on the strain gauge readings, the lateral strain of the cores tended to be larger at the ultimate load, compared to Specimens MC1-MC6. The lateral strain of concrete cores increased rapidly after the ultimate load. By reaching 95% beyond the peak load, the cores expanded significantly, while the strength degradation was much less than that of Specimens MC1-MC6.

7.12.3 Steel, Wire Mesh and MHSM Confined Specimens

7.12.3.1 Ultimate Load Estimation

In Experiment III, steel reinforcement, wire mesh and MHSM composite was adopted as confinement. The geometry of cross section is shown in Fig. 7.33. The lateral pressure exerted by circular steel hoops and closely spaced lateral wires can be computed from statics by computing lateral pressure separately for each type of confinement then superimposing the two types of confinement. The assumptions are: 1) an equivalent uniform pressure was applied to the core; 2) the cross section of strengthened specimens remains plane after deformation; 3) there is no sliding between the concrete core and the confinement composite; 4) the lateral wires
yielded at the ultimate load; 5) the contribution of vertical wires is negligible; and 6) the gain on the strength of the mortar inside the cage was not taken into account as the confining pressure contributed by lateral wires is small, and also for simplicity.

![Fig. 7.33 Cross section of steel, wire mesh and MHSM confined specimens](image)

In the case of circular lateral reinforcement, the reduction of the core area to an effectively confined area takes place only along the longitudinal axis of the column (Mander et al. 1988a; Saatcioglu and Razvi 1992; Cusson and Paultre 1995). Based on AS3600 (2009), the effectiveness factor was calculated for each type of reinforcement and presented in Table 7.23. The effectiveness factor for wire mesh confinement is relatively high due to small aperture. Installing wire mesh tube can enhance the effectively confined area along the vertical axis and provide additional confining pressure at the hoop level. With the increase in the number of mesh layers and the mortar strength, the stiffness of the confined core is also improved.

<table>
<thead>
<tr>
<th>Spacing (mm)</th>
<th>Steel Hoop $k_{ex}$</th>
<th>Wire Mesh $k_{ew}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>0.71</td>
<td>1</td>
</tr>
<tr>
<td>75</td>
<td>0.58</td>
<td>2</td>
</tr>
<tr>
<td>100</td>
<td>0.46</td>
<td>3</td>
</tr>
</tbody>
</table>

As mentioned in Chapter 5, the volumetric ratio ($\rho_s$), for the specimens with steel hoops spaced at 100 mm, 75 mm and 50 mm, was 0.72%, 0.97% and 1.45%,
respectively. This range of volumetric ratios is considered as low to modest for spirally confined NSC columns (Sheikh and Toklucu 1993). The volumetric ratios of lateral wire were calculated based on (AS 3600-2009). The details for the volumetric ratios of each type of confinement are summerised in Table 7.24.

<table>
<thead>
<tr>
<th>Steel Reinforcement Type</th>
<th>Steel Hoop Spacing (mm)</th>
<th>Steel Volumetric Ratio $\rho_s$ (%)</th>
<th>Wire mesh, hoop Volumetric Ratio $\rho_{s+1M}$ (%)</th>
<th>$\rho_{s+2M}$ (%)</th>
<th>$\rho_{s+3M}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wire mesh and Steel hoop</td>
<td>100</td>
<td>0.72</td>
<td>0.91</td>
<td>1.09</td>
<td>1.25</td>
</tr>
<tr>
<td></td>
<td>75</td>
<td>0.97</td>
<td>1.16</td>
<td>1.34</td>
<td>1.50</td>
</tr>
<tr>
<td>Steel hoop</td>
<td>50</td>
<td>1.45</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

As can be seen in Table 7.24, the lateral reinforcement ratio significantly increased due to wires mesh contribution. According to ACI 318-08, the required volumetric ratio of spiral for non-seismic design can be calculated using Eq. 7.10. It is noted that the compressive strength of concrete was replaced by the compressive strength of mortar. The yield stress of steel based on 0.2% standard strain was adopted, as indicated in Section 6.4, Chapter 6.

$$\rho_s = 0.45 \left( \frac{A_s}{A_c} - 1 \right) \frac{f'_c}{f_{ys}}$$  \hspace{1cm} (7.10)

If alternately using spiral, the minimum of volumetric spiral reinforcement ratio would be 2.6%, 2.0%, 2.5%, 2.3%, 2.5% and 2.6% for FMB1-FMB6, respectively. The amount of transverse reinforcement affects the stability of reinforcement tube, which is essential for continuously providing effective confinement against the lateral expansion of concrete (Saatcioglu and Razvi 1992). In this study steel hoops with relatively low volumetric ratios were adopted to examine the effect of wire mesh.

The confining pressure contributed by wire mesh was obtained using Eq.7.3 and Eq.7.4, as defined earlier; the confining pressure contributed by steel hoops was determined using Eq7. 11. The strength of confined core was calculated using Eq. 7.12.
where $f_{lw}$ and $f_{ls}$ are the confining pressure contributed by wire mesh and steel hoops, respectively; $k_1 = 4.1$; $f_s$ is the stress of steel hoops based on measurement at the second peak load or closely past the second peak load on the plateau and the average stress was 542 MPa, approximately 82% of the yield stress. All the other notations are as indicated previously. As the compressive strength of MHSM was high, the load carried by MHSM inside the confinement cage is considered up to the mean of the mesh layers. Otherwise, the strength of the confined core could be significantly overestimated, particularly for the case where the confinement cage is large.

A number of models, as shown in Table 7.25, were adopted for predicting the ultimate load of steel hoops, wire mesh and MHSM confined specimens. It is noted that the model proposed by Samaan et al. (1998) is based on GFRP tube. The average steel stress (542 MPa) based on measurement was used for predicting the peak load, noted as $P_2$. The two methods of mortar infill and mortar plate as mentioned earlier are adopted. The mortar infill method is presented as Eq. 7.13, in which the contribution of mortar in the reinforcement tube is taken into account.

\[
 f'_{sw} = f'_{sw} + k_1 (k_{cw} f'_{w} + k_{cw} f'_{w})
\]

(7.12)

\[
f_s = 1 - \frac{s}{2D_h} \left( \frac{2A_h f_s}{D_s s} \right)
\]

(7.11)

\[
 P_2 = A \left( f'_{sw} + k_1 (k_{cw} f'_{w} + k_{cw} f'_{w}) \right) + 0.85 \alpha_{1,m} f'_{m} \pi (D^2_w - D^2_t) / 4
\]

(7.13)

The notations are as defined previously. It is noted that 0.85 is the strength reduction factor. Due to the presence of steel hoops and wire mesh tube, the strength of MHSM was further affected by the number of mesh layers and workmanship. Therefore, it is reasonable to employ a reduction factor of 0.85. The product of the reduction factor and $\alpha_{1,m} = 1-0.003 f'_{m}$ ranged from 0.69 to 0.73. Other researchers adopted one factor with a value of 0.67 for determining the compressive strength of mortar (Mansur and Paramasivam 1990).
For the method of mortar plate, the confining pressure due to ferrocement was obtained using Eq. 7.2, based on the experiment results of the ultimate tension load of ferrocement plates. Refer to Chapter 6 for the test results of the ultimate tension load. Accordingly, the ultimate load $P_2$ can be presented as following (all the notations are as indicated previously):

$$P_2 = A \left( f_{w2} + k_1 \left( k_{w2} f_{w2}' + f_{w2}'' \right) \right) + 0.85 \alpha f_{w2} \pi \left( D_w^2 - D_2^2 \right) / 4 \quad (7.14)$$

The same two statistic indicators MSE and AAE were used to evaluate the prediction accuracy. For the 18 composite tube confined specimens; the values of MSE and AAE for each model are presented in Table 7.25 and Fig. 7.34.
Table 7.25 Estimation of the ultimate load ($P_2$) for steel, wire mesh and MHSM confined specimens

<table>
<thead>
<tr>
<th>No.</th>
<th>Model</th>
<th>Expression</th>
<th>Ultimate Load</th>
<th>MSE (%)</th>
<th>AAE (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Mortar Infill</td>
<td>$P_2 = A_s \left( f_{co} + k_c (k_m f_{s} + k_m f_{w}^i) \right) + 0.85\alpha f_m^s \pi \left( D_m^i - D_s^i \right) / 4$</td>
<td>0.58</td>
<td>6.36</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Mortar Plate</td>
<td>$P_2 = A_s \left( f_{co} + k_c (k_m f_{s} + f_{w}^i) \right) + 0.85\alpha f_m^s \pi \left( D_m^i - D_s^i \right) / 4$</td>
<td>0.61</td>
<td>7.07</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Kaish et. al 2015</td>
<td>$P_2 = A_s \left( f_{co} + 11.2 k_m f_{w}^i + 11.2 k_m f_{w}^i \right)$</td>
<td>1.16</td>
<td>8.48</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Xiong et al. 2011</td>
<td>$P_n^x = A_s f_{co} (1 + 1.98\lambda_s)$</td>
<td>2.48</td>
<td>13.15</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Rechart et al. 1928</td>
<td>$P_2 = A_s \left( f_{co} + 4.1 (k_m f_{s} + k_m f_{w}^i) \right)$</td>
<td>4.34</td>
<td>19.80</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Balmer 1949</td>
<td>$P_2 = A_s f_{co} \left( 1 + 9.175 \left( k_m f_{s}^i + k_m f_{w}^i \right) / f_{co} \right)^{0.73}$</td>
<td>1.71</td>
<td>10.97</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Fafitis and Shah 1985</td>
<td>$P_n = A_s \left( f_{co} + \left( 1.15 + \frac{21}{f_{co}} \right) (k_m f_{s} + k_m f_{w}^i) \right)$</td>
<td>31.22</td>
<td>9.95</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Mander et al. 1988a</td>
<td>$P_n = A_s \left( f_{co} 2.254 \sqrt{1 + 7.94 \left( k_m f_{s}^i + k_m f_{w}^i \right) / f_{co}} - 2(k_m f_{s}^i + k_m f_{w}^i) - 1.254 f_{co} \right)$</td>
<td>1.86</td>
<td>12.55</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Saatcioglu and Razvi 1999</td>
<td>$P_n = A_s \left( f_{co} + 6.7 \left( k_m f_{s} + k_m f_{w}^i \right)^{0.83} \right)$</td>
<td>2.15</td>
<td>12.91</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Samaan et al.1998</td>
<td>$P_n = A_s \left( f_{co} + 6 \left( k_m f_{s} + k_m f_{w}^i \right)^{0.7} \right)$</td>
<td>3.82</td>
<td>18.71</td>
<td></td>
</tr>
</tbody>
</table>
Fig. 7.34 Evaluation of models for steel, wire mesh and MHSM confined specimens

As shown in Table 7.25, the proposed two methods had significant less value in both AAE and MSE. For each approach, the variation was large for the first and second group specimens, except for Specimens 1M502, 2M50S1 and 2M50S2. For the third group, the variation for Specimens 3M100S1 and 3M100S2 was also large, but for Specimens 3M75S1, 3M75S2, 3M50S1 and 3M50S2 it was significantly small. This indicates that at the ultimate load, the effectiveness of confinement was different among the steel, wire mesh and MHSM reinforced specimens, evidenced by the axial load and axial displacement curves. Accordingly, different approach needs to be adopted. For the case that the load carrying capacity was mainly contributed by cementitious matrices, Eq. 7.15 can be used to determine the ultimate load ($P_1$).

$$P_1 = f_{\text{cm}} A_e + 0.85 \alpha_{\text{cm}} f_{\text{cm}}' (D^3 - D_e^3) / 4$$  \hfill (7.15)

All the notations are as defined previously. Based on Eq. 7.15, the comparison between the estimated and the experimental results is illustrated in Fig. 7.35, including all the steel, wire mesh and MHSM confined specimens. As no longitudinal reinforcement was applied and steel volumetric ratio was relatively low, Eq. 7.15 provides an approximate estimation for all the steel, wire mesh and MHSM
confined specimens. The values of MSE and AAE of each method for Specimens 1M100S1, 1M100S2, 1M75S1, 1M75S2, 1M50S1, 1M50S2, 2M100S1, 2M100S2, 2M75S1, 2M75S2, 3M100S1 and 3M100S2 are illustrated in Fig. 7.36.

![Fig. 7.35 P₁ Estimation based on Eq. 7.15](image)

![Fig. 7.36 MSE and AAE for specimens with ineffective confinement at P₀](image)

As illustrated in Fig. 7.36, for the case of ineffective confinement at the ultimate load, Eq. 15 (P₁) had reasonably close agreement with the experimental results, evidenced by the AAE value of 5.1%. It does not only provide numerical estimation, but also consists with the failure mode.
The confinement models were adopted to estimate the ultimate load \( (P_2) \) for the case of the presence of the second peak load, including 1M50S2, 2M50S1, 2M50S2, 3M75S1, 3M75S2, 3M50S1 and 3M50S2. The estimated load and the strength of the confined core are summarised in Table 7.26.

The comparison of these models is illustrated in Fig. 7.37. This figure shows that in terms of the strength of confined core, the models that had relatively close approximation to the test results are Sammaan et al. (1998), Mortar Infill, Mortar Plate and Saatcioglu and Razvi (1999); in terms of the peak load, Kaish et al. (2015), Balmer (1949), Mander et al. (1989 a) and Saatcioglu and Razvi (1999) indicate relatively accuracy.

The experimental strength of the confined concrete cores were obtained by subtracting the contribution of mortar in the reinforcement tube from the second peak load or closely past the second peak load on the plateau, where the steel stress was approximately 542 MPa based on strain measurement, and then dividing by the core area. At that point the protective cover was nearly spalled off or loose. The specimens undertook large deformation both in the axial and the lateral directions.

As mentioned earlier, if the contribution of mortar in the confinement tube was not subtracted, it would lead to significant overestimation. For example, by subtracting the contribution of MHSM, the estimated strength of the confined core of Specimens 3M75S1 and 3M75S2, was 46 MPa, which otherwise could be 58 MPa. To achieve this strength, the same R6 steel hoops would be spaced at 30 mm.
Table 7.26 Strength and peak load of specimens with relatively higher volumetric ratio

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Experiment</th>
<th>Mortar Infill</th>
<th>Mortar Plate</th>
<th>No.3 Table 7.25</th>
<th>No.4 Table 7.25</th>
<th>No.6 Table 7.25</th>
<th>No.8 Table 7.25</th>
<th>No.9 Table 7.25</th>
<th>No.10 Table 7.25</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$f_{cc}$</td>
<td>$P$</td>
<td>$f_{cc}$</td>
<td>$P$</td>
<td>$f_{cc}$</td>
<td>$P$</td>
<td>$f_{cc}$</td>
<td>$P$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>kN</td>
<td>MPa</td>
<td>kN</td>
<td>MPa</td>
<td>kN</td>
<td>MPa</td>
<td>kN</td>
<td>MPa</td>
</tr>
<tr>
<td>1M50S2</td>
<td></td>
<td>952</td>
<td>47</td>
<td>51</td>
<td>1021</td>
<td>55</td>
<td>962</td>
<td>59</td>
<td>1163</td>
</tr>
<tr>
<td>2M50S1</td>
<td></td>
<td>1035</td>
<td>48</td>
<td>53</td>
<td>1107</td>
<td>61</td>
<td>1067</td>
<td>65</td>
<td>1275</td>
</tr>
<tr>
<td>2M50S2</td>
<td></td>
<td>994</td>
<td>47</td>
<td>53</td>
<td>1097</td>
<td>61</td>
<td>1067</td>
<td>65</td>
<td>1275</td>
</tr>
<tr>
<td>3M75S1</td>
<td></td>
<td>1028</td>
<td>46</td>
<td>49</td>
<td>1080</td>
<td>61</td>
<td>1068</td>
<td>65</td>
<td>1278</td>
</tr>
<tr>
<td>3M75S2</td>
<td></td>
<td>1031</td>
<td>46</td>
<td>49</td>
<td>1086</td>
<td>61</td>
<td>1068</td>
<td>65</td>
<td>1278</td>
</tr>
<tr>
<td>3M50S1</td>
<td></td>
<td>1127</td>
<td>51</td>
<td>55</td>
<td>1180</td>
<td>66</td>
<td>1168</td>
<td>71</td>
<td>1395</td>
</tr>
<tr>
<td>3M50S2</td>
<td></td>
<td>1101</td>
<td>50</td>
<td>55</td>
<td>1186</td>
<td>66</td>
<td>1168</td>
<td>71</td>
<td>1395</td>
</tr>
</tbody>
</table>

Note: $P^*$ is the second peak load or closely past the second peak load.

![Load estimation](a)

![Strength of confined core, $f_{cc}$](b)

Fig. 7.37 Comparison of Models
The estimated and experimental values of the strength of confined cores are illustrated in Fig. 7.38, which indicates that the estimated values are larger than the experimental ones. The increase in the compressive strength of concrete cores is plotted as a function of effective confinement pressure, as shown in Fig. 7.39. A best-fit line is given as Eq. 7.16. It is noted that \( f_i \) is the combined lateral pressure contributed by steel hoops and wire mesh.

\[
f_{cc} - f_{co} = 3.43 \left( k_o f'_s + k_v f'_w \right)
\]  
(7.16)
As mentioned earlier, spiral or steel hoops might be considered as a kind of steel tube when pitch or spacing is close enough (Guo and Shi 2003). In light with the equivalent continuous jacket thickness for steel reinforcement (Binici 2005), the equivalent continuous jacket thickness for steel hoops and wire mesh confined concrete can be calculated using Eq. 7.17 and Eq. 3.9, respectively. Refer to Section 3.17 for the details of determining the equivalent continuous jacket thickness of wire mesh.

\[ t_{js} = \frac{A_s}{s} \left( 1 - \frac{s}{\sqrt{1.25D_c}} \right) \]  

(7.17)

where \( t_{js} \) is the thickness of the equivalent continuous jacket attributed by steel hoops; \( A_s \) is the cross area of steel hoops; \( s \) and \( D_c \) are as defined previously. As mentioned earlier, if the spacing of spirals is larger than the distance equal to 1.25 times the diameter of the confined core, the effect of confinement can be considered negligible (Ahmad and Shah 1982).

For steel tube confinement, the reinforcement ratio, \( \mu_{ts} \), can be determined using Eq. 7.18, as the tube thickness is considerably less than the diameter of confined concrete (\( t_{js} \ll D_c \)). The confinement ratio can be determined using Eq. 7.19 (Guo and Shi 2003). The notations are as identified previously.

\[ \mu_{ts} = \frac{4t_{js}}{D_c} \]  

(7.18)

\[ \lambda_w = \frac{4t_{js}f_s}{D_c f_{yw}} = \mu_{ts} \frac{f_s}{f_{yw}} \]  

(7.19)

The equivalent thickness due to wire mesh was calculated using Eq. 3.12; for one layer, two layers and three layers of wire mesh, the equivalent thickness is 0.06 mm, 0.12 mm and 0.18 mm, respectively. The ultimate load can be approximately calculated using Eq. 7.20. It is noted that \( \lambda_{total} \) is the sum of \( \lambda_{tw} \) and \( \lambda_{ts} \). The notations are as identified previously. The results are presented in Table 7.27.
\[ P_2 = f_{\text{cm}}(1 + 2\lambda_{\text{total}})A_e + 0.85\alpha_{\text{cm}}f_{\text{cm}}'(D_e^2 - D_i^2) / 4 \] (7.20)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Tested Load (P^*) (kN)</th>
<th>Predicted Load (P_{\text{mod}}) (kN)</th>
<th>Deviation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1M50S2</td>
<td>952</td>
<td>946</td>
<td>-0.6</td>
</tr>
<tr>
<td>2M50S1</td>
<td>1035</td>
<td>1025</td>
<td>-1.0</td>
</tr>
<tr>
<td>2M50S2</td>
<td>994</td>
<td>1015</td>
<td>2.2</td>
</tr>
<tr>
<td>3M75S1</td>
<td>1028</td>
<td>1016</td>
<td>-1.2</td>
</tr>
<tr>
<td>3M75S2</td>
<td>1031</td>
<td>1021</td>
<td>-0.9</td>
</tr>
<tr>
<td>3M50S1</td>
<td>1127</td>
<td>1091</td>
<td>-3.2</td>
</tr>
<tr>
<td>3M50S2</td>
<td>1101</td>
<td>1097</td>
<td>-0.4</td>
</tr>
<tr>
<td>AAE (%)</td>
<td>-</td>
<td>-</td>
<td>1.47</td>
</tr>
<tr>
<td>MSE (%)</td>
<td>-</td>
<td>-</td>
<td>0.03</td>
</tr>
</tbody>
</table>

Note: \(P^*\) is the second peak load or closely past the second peak load.

The results indicate that the equivalent steel tube method provides good agreement with the experiment results. This is probably because with the decrease in the steel hoop spacing and the increase of the number of mesh layers, the confinement cage is similar to steel tube. The mortar infill is considered up to the mean of the mesh layers, \(D_m\). The deviations between the estimated loads and the experimental results based on mortar plate, mortar infill, equivalent steel tube and modified Samaan et al. (1998) are plotted in Fig. 7.40. The steel tube analogy was also adopted for the specimens externally confined with wire mesh in Chapter 3. With the increase in the reinforcement volumetric ratio, the equivalent steel tube model performs better. The estimation based on the modified Samaan et al. (1998)’s approach also had relatively good agreement.

To sum up, for the case that the contribution of lateral reinforcement is less than the load carried by cover, the ultimate load is mainly contributed by the cementitious matrices, noted as \(P_l\) and can be estimated using Eq. 7.15. For the case that the lateral confinement is effective, the ultimate load can be estimated using Eq. 7.13, which provides reasonable agreement compared with the confinement models discussed in the previous section. In addition, the method of equivalent steel tube with mortar infill, shown as Eq. 7.20, provides the estimation with good agreement. It is noted
that: 1) the stress in steel reinforcement may not reach the yield stress at the second peak load; 2) MHSM in the reinforcement tube needs to be considered.

It is noted that the existing research (Bazant and Xi 1991; Elfahal 2003) has shown that size effect may affect the structural behaviour; Elfahal (2003) concluded that the size effects can be characterised in terms of the nominal stress at failure, concrete modulus of elasticity and strain at maximum stress. However, such effects are believed to be negligible for concrete structures under a relatively high level of confinement (Kim et al. 1999; Carey and Harries 2005; Matthys et al. 2005).

Therefore, for the present study, it is reasonable to consider the size effect is negligible for the strengthened members with a relatively high level of confinement (steel hoops spaced at 75 mm or 50 mm with three layers of mesh); while for the weakly-confined specimens (steel hoops spaced at 100 mm with wire mesh), the size effect probably needs to be considered and further research is needed for clarification.
Fig. 7.40 Deviation of selected methods
7.12.3.2 Cover Thickness

Various cover thickness were adopted for applying wire mesh, ranged from 2 mm to 25 mm (Takiguchi K. and Abdullah 2001; Hadi and Zhao 2011; Xiong et al. 2011; Ho et al. 2013). In Experiment III, the protective cover thickness varied from 14 mm to 5.5 mm. As mentioned in Chapter 5, identical moulds were used for mortar casting, except for Specimens 3M100S1, 3M100S2, 3M75S1, 3M75S2, 3M50S1 and 3M50S2. For these six specimens, mortar was manually packed with a specially curved trowel with a radius of 95 mm and 5 mm thick plastic strips were used to control the cover thickness. The measured $D_w$ was 179 mm for the specimens confined with steel hoops and three layers of wire mesh. Therefore, the nominal thickness was 5.5 mm.

The maximum thickness of protective cover of the steel, wire mesh and MHSM reinforced specimens was calculated by setting the total confinement effect of steel reinforcement and wire mesh not less than the load carried by the protective cover, as shown in Eq. 7.21 and Eq. 7.22. The calculated protective cover thicknesses are presented in Table 7.28.

\[
3.43 \left( k_{ew} \frac{2nA_{mesh}f_{yw}}{D_wS_w} + k_{es} \frac{2A_s f_s}{D_s} \right) A_c \geq 0.85 \alpha_{1m} f_{yw} \pi \left( \left( D_w + 2t \right)^2 - D_w^2 \right) / 4 \tag{7.21}
\]

\[
0.85 \alpha_{1m} f_{yw} t^2 + 0.85 \alpha_{1m} f_{yw} D_w t - 0.86 \left( k_{ew} \frac{2nA_{mesh}f_{yw}}{D_wS_w} + k_{es} \frac{2A_s f_s}{D_s} \right) D_c^2 \leq 0 \tag{7.22}
\]

where $D_w$ = the overall dimension measured between the outermost wires; $D_m$ = the overall dimension of the mean of the wire mesh layers; $D_m = (D_e + D_w)/2$; $k_{ew}$, $k_{es}$ = as defined previously.
Table 7.28 Cover thickness of steel, wire mesh and MHSM confined specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Calculated cover (mm)</th>
<th>Actual cover (mm)</th>
<th>Mould for cover</th>
<th>Ultimate load $P_u$ (kN)</th>
<th>Plateau or $P_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1M100S1</td>
<td>3.7</td>
<td>11</td>
<td>Y</td>
<td>1009</td>
<td>N</td>
</tr>
<tr>
<td>1M100S2</td>
<td>4.5</td>
<td>11</td>
<td>Y</td>
<td>914</td>
<td>N</td>
</tr>
<tr>
<td>1M75S1</td>
<td>5.2</td>
<td>11</td>
<td>Y</td>
<td>1100</td>
<td>N</td>
</tr>
<tr>
<td>1M75S2</td>
<td>6.4</td>
<td>11</td>
<td>Y</td>
<td>919</td>
<td>N</td>
</tr>
<tr>
<td>1M50S1</td>
<td>8.3</td>
<td>11</td>
<td>Y</td>
<td>1176</td>
<td>N</td>
</tr>
<tr>
<td>1M50S2</td>
<td>10.1</td>
<td>11</td>
<td>Y</td>
<td>958</td>
<td>Y</td>
</tr>
<tr>
<td>2M100S1</td>
<td>5.0</td>
<td>8.5</td>
<td>Y</td>
<td>1096</td>
<td>N</td>
</tr>
<tr>
<td>2M100S2</td>
<td>5.2</td>
<td>8.5</td>
<td>Y</td>
<td>1055</td>
<td>N</td>
</tr>
<tr>
<td>2M75S1</td>
<td>6.5</td>
<td>8.5</td>
<td>Y</td>
<td>1025</td>
<td>N</td>
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<tr>
<td>2M75S2</td>
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<td>8.5</td>
<td>Y</td>
<td>1061</td>
<td>N</td>
</tr>
<tr>
<td>2M50S1</td>
<td>9.6</td>
<td>8.5</td>
<td>Y</td>
<td>1076</td>
<td>Y</td>
</tr>
<tr>
<td>2M50S2</td>
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<td>8.5</td>
<td>Y</td>
<td>1034</td>
<td>Y</td>
</tr>
<tr>
<td>3M100S1</td>
<td>5.8</td>
<td>5.5</td>
<td>N</td>
<td>1152</td>
<td>N</td>
</tr>
<tr>
<td>3M100S2</td>
<td>5.7</td>
<td>5.5</td>
<td>N</td>
<td>1038</td>
<td>N</td>
</tr>
<tr>
<td>3M75S1</td>
<td>7.2</td>
<td>5.5</td>
<td>N</td>
<td>1028</td>
<td>Y</td>
</tr>
<tr>
<td>3M75S2</td>
<td>7.1</td>
<td>5.5</td>
<td>N</td>
<td>1031</td>
<td>Y</td>
</tr>
<tr>
<td>3M50S1</td>
<td>10.1</td>
<td>5.5</td>
<td>N</td>
<td>1180</td>
<td>Y</td>
</tr>
<tr>
<td>3M50S2</td>
<td>9.9</td>
<td>5.5</td>
<td>N</td>
<td>1101</td>
<td>Y</td>
</tr>
<tr>
<td>STL50S1</td>
<td>9.2</td>
<td>14</td>
<td>Y</td>
<td>972</td>
<td>N</td>
</tr>
<tr>
<td>STL50S2</td>
<td>9.0</td>
<td>14</td>
<td>Y</td>
<td>1093</td>
<td>N</td>
</tr>
</tbody>
</table>

Three situations regarding the calculated cover and the actual cover thickness were identified. For the first situation, if the calculated cover thickness is significantly less than the actual protective cover, the increment in strength due to confinement cannot balance the cover loss. The ultimate load was reached with a relatively small axial displacement. No load plateau or a second peak load appeared. This situation was illustrated by the specimens of the first group, except for Specimen 1M50S2. Specimen 1M50S2 reached the first peak load (maximum load) with a small axial displacement. However, a long load plateau appeared with a second peak load, which was almost the same as the first peak. For the second situation, if the calculated cover thickness is significantly greater than the actual one, the load due to cover loss can be balanced by the gain in strength due to the confinement. A second peak load appeared in Specimen 3M50S1 and Specimen 3M50S2. For the third situation, if the calculated cover thickness is only slightly greater than the actual one, for example
Specimens 2M50S1, 2M50S2, 3M75S1 and 3M75S2, a second peak load, which is slightly less than the first peak load or a load plateau with a large axial displacement appeared. It is noted that the calculated cover thicknesses for Specimens 3M100S1 and 3M100S2 were marginally greater than the actual values, and no second load or load plateau was presented. This might be related to the circumference tolerance. In general, the larger the calculated cover was than the actual cover, the more ductile behavior occurred.

Due to relatively low volumetric ratio and high strength mortar, whether reinforcement becomes effective is sensitive to the cover thickness. In general, less protective cover for wire mesh confined members is preferable. In addition to the feasibility of cover control, a cover with 5 mm thickness can protect wires and enhances a better performance. A relatively thin cover enables wires to be evenly distributed in the matrix and increases the specific surface of reinforcement ($S_r$), resulting in the decrease in the cracking spacing and better energy absorption. The procedure for determining the cover thickness and the ultimate load of steel hoops, wire mesh and MHSM confined concrete is illustrated in Fig. 7.41.
7.13 Conclusions

Experimental studies were carried out to investigate the axial compressive behaviour of NSC confined with different types of materials. Based on the results analysis, the following conclusions are drawn:
Unlike HSC cover spalling, which occurred in a sudden manner and was relatively easy to identify, the MHSM cover was unloaded gradually due to the presence of wire mesh. With the increase of the number of mesh layers (fracture ratio, $V_f$), the cover spalled more (refer to Fig. 7.3 for details), but in a gradual manner;

Specimens confined with three layers of wire mesh only ($\rho_{3M} = 0.59\%$) can perform comparably to or potentially better than those with low steel volumetric ratio ($\rho_s = 0.72\%$);

Due to the application of wire mesh, there was clear and significant improvement in ductility. Ductility analysis show that specimens confined with relatively low steel volumetric ratio ($\rho_s = 0.97\%, \rho_s = 1.45\%$) and combined with three layers of wire mesh ($\rho_w = 0.53\%$) can achieve ductility levels comparable to those well-confined concrete, evidenced by a long load plateau or a second peak load;

High strength mortar matrices have significant influence on the ultimate load. Compared to Specimens MC1-MC6, there was no improvement in the ultimate load for nearly all the specimens confined with steel hoops, wire mesh and MHSM except for Specimen 3M50S1 showing a slight increment; while for the wire mesh and MHSM confined specimens, the ultimate load did not increase;

For specimens confined with wire mesh and MHSM, the ultimate load was mainly attributed by cementitious matrices. For steel hoops, wire mesh and MHSM confined specimens, influenced by the reinforcement ratio, mortar strength and cover thickness, steel reinforcement became effective at the second peak load or shortly past the second peak load, but not fully active;

Generally, the yield stress of transverse steel reinforcement is used for determining the confining pressure. The yield stress of transverse steel reinforcement is specified not greater than 800 MPa in AS3600 (2009) and 700 MPa in ACI 318 (2008). However, the observation of this study confirms that lateral steel confinement did not reach yield stress at the second peak load despite NSC core;

The proposed Eq. 7.1, Eq. 7.13, Eq. 7.15 and Eq. 7.20 provide estimations for different cases with reasonable accuracy;
Experiments have shown that with relatively low volumetric ratio, the effect of confinement on the load carrying capacity of confined members is limited. For the case of no longitudinal steel reinforcement and low (or relatively low) reinforcement volumetric ratio, practically the estimation of \( P_t \) may be used as an approximation for the load carrying capacity, as shown in Fig. 7.42, which includes all the experiment data and some data from the literature. It is noted that the two dash lines are closely parallel with the line of perfect prediction, which has a slope of 1 and passing through the origin. This figure shows that the test results of the different series of specimens exhibited reasonable scatter;

![Fig. 7.42 Estimated \( P_t \) of three experiments](image)

Wire mesh has good energy absorption capacity. Steel hoops have more significant influence on energy absorption. However, two and three layers of wire mesh made comparable contribution, compared to specimens confined with low steel reinforcement ratio. The energy absorption capability increases with the increase of \( V_f \) and \( \rho_s \);

At relatively low reinforcement volumetric ratio (\( \rho_s \)), steel reinforcement, wire mesh and MHSM composite can employ the advantage of each material and significantly improve ductility.
CHAPTER 8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Introduction

This thesis presents a systematic study on the utilisation of wire mesh, transverse steel and modified high strength mortar composite for the strengthening and/or repair of concrete members. While the PhD study was limited to the uniaxial compressive behaviour, it provides a solid basis for future research on the behaviour of such strengthened RC structures under combined axial load and bending. The research outcomes of the present study can be directly applied to, or be extended for, actual RC columns. Based on the series of experimental results and analysis, the conclusions are drawn in the last chapter. Additionally, recommendations for further research are provided at the end.

8.2 Conclusions

Based on the investigation on applying wire mesh to HSC, MSC and NSC, the main findings from the present study are drawn as follows:

8.2.1 Strengthening HSC with Wire Mesh and High Strength Mortar

(1) In general, unlike HSC cover spalling, which occurred in a sudden manner and relatively easy to identify, high strength mortar cover was unloaded gradually. This is due to the uniform dispersion of wires, which transforms the brittle mortar into a material with good ductility;

(2) By using wire mesh and high strength mortar with a 5 mm or 10 mm thick cover, the ultimate load improved significantly. However, the increment in the load carrying capacity was likely due to the contribution of concrete and mortar. Wire mesh became effective during the post peak stage;

(3) The amount of reinforcement ($V_f = 0.19\%$, $V_f = 0.38\%$) cannot provide the minimum effective confining pressure required by AS3600 (2009). However, it was sufficient to change the typical HSC failure mode and the ductility of wire mesh strengthened specimens was improved modestly.
8.2.2 Cast MSC with Wire Mesh

(4) For the MSC specimens internally reinforced with wire mesh, the ultimate load increased marginally, while ductility increased significantly. The effect of wire mesh was pronounced on ductility than on the load carrying capacity;

(5) Contrast to what was commonly reported in the literature, the increment in the number of mesh layers did not ensure the increase in the ultimate load, but markedly affected the post peak behaviour of confined specimens;

(6) Due to the nature of wire mesh, whether wire mesh becomes effective at the ultimate load is sensitive to the cover thickness. The load carrying capacity can be estimated based on the contribution of matrices (Eq. 3.8) with reasonable accuracy;

(7) For the specimens externally confined with wire mesh and no cover applied, the ultimate load increased significantly, while ductility improved modestly.

8.2.3 Strengthening NSC with either Wire Mesh and MHSM or Steel Hoops, Wire Mesh and MHSM Composite

(8) A composite of steel hoops, wire mesh and MHSM can employ the advantages of each material, enhance the load carrying capacity, and significantly improve ductility. Steel confinement has a primary influence on the axial compression behaviour of the strengthened specimens in terms of strength, ductility, first steel hoop fracture, energy absorption and failure mode;

(9) MHSM has significant influence on the load carrying capacity. For the specimens strengthened with wire mesh and MHSM, the ultimate load was mainly attributed by matrices. For the steel hoops, wire mesh and MHSM strengthened specimens, steel reinforcement became effective at the second peak load, but not fully active;

(10) Generally, the yield stress of transverse steel reinforcement is used for determining the confining pressure. However, the observation of this study confirms that lateral steel confinement did not reach the yield stress at the second peak load; The empirical equations Eq. 7.1, Eq. 7.13, Eq. 7.15 and Eq. 7.20 provide estimations for different cases with reasonable accuracy;
As a secondary confinement, a sufficient amount of wire mesh can facilitate restraining and uniformly dispersing the dilation of steel hoops. Therefore, the confinement effect is enhanced. Wire mesh markedly enhances ductility. Ductility analysis shows that specimens strengthened with steel hoops spaced at 75 mm ($\rho_s = 0.97\%$), 50 mm ($\rho_s = 1.45\%$) with three layers of mesh achieved a ductility level comparable to those well-confined concrete, evidenced by a desirable long load plateau or a second peak load;

Specimens confined with sufficient amount of wire mesh only ($\rho_w = 0.59\%$) is likely to perform comparably to those with a low transverse steel volumetric ratio ($\rho_s = 0.72\%$). Further research is needed to investigate the equivalent ratios between $\rho_w$ and $\rho_s$;

Wire mesh has good energy absorption capacity, which generally increases with the increase of $\nu'$.  

8.3 Recommendations for Further Study

Based on the present work, the following recommendations are suggested for further studies on this concrete strengthening and/or repair technique:

1. In general, less protective cover for wire mesh confined members is preferable. In addition to the feasibility of cover control, a protective cover of 5 mm balances the protection to wires and enhances a better performance. This enables wires to be distributed in the matrix relatively evenly and increases the specific surface of reinforcement ($S_r$), resulting in smaller cracking spacing and improving energy absorption;

2. Three layers of wire mesh is recommended for significant improvement;

3. Experiment instrumentation can be improved to obtain the complete axial strain of the confined specimens;

4. A new method or improvement in measuring the strain in steel reinforcement is required due to the constraint of wire mesh tube;

5. The steel hoops, wire mesh and MHSM composite can be applied for strengthening HSC members under concentric loading and eccentric loading. The reinforcement volumetric ratio needs to be increased accordingly due to
the nature of HSC. For the case of eccentric loading, both the trial-error procedure of the conventional reinforced concrete and the rigid-plastic concept can be applied for analysing the bending-axial load interaction behaviour;

(6) As wire mesh has the ability to undergo large deformation, further research can be extended to the effect of the steel, wire mesh and MHSM composite under cyclic loading;

(7) In addition to strengthen concrete members, steel hoops, wire mesh and MHSM composite may also be used to change the geometry of the section for further strengthening with other materials;

(8) As a useful structural engineering tool, numerical analysis has been used for analysing complex structures and for parametric study. Currently, finite element analysis on ferrocement-confined concrete columns is very limited (Kaish et al. 2018). To optimise the application of wire mesh, numerical analysis needs to be conducted not only on ferrocement confined plain or reinforced concrete columns but also the combination of wire mesh and steel reinforcement as well as high strength mortar strengthened members for different loading, including axial force, bending and shear.


ACI Committee 318 (2008), Building Code Requirements for Structural Concrete (ACI 318-14), American Concrete Institute, Farmington Hills, Michigan.


