Engineering properties and flexural performance of carbon nanofibers enhanced lightweight cementitious composite (CNF-LCC)

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SINGAPORE

**Engineering Properties And Flexural Performance
of Carbon Nanofibers Enhanced Lightweight
Cementitious Composite (CNF-LCC)**

WANG SU

School of Civil and Environmental Engineering

2019

**Engineering Properties And Flexural Performance
of Carbon Nanofibers Enhanced Lightweight
Cementitious Composite (CNF-LCC)**

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School of Civil and Environmental Engineering

A thesis submitted to the Nanyang Technological University

in partial fulfillment of the requirements for the degree of

Doctor of Philosophy

2019

Statement of Originality

I hereby certify that the work embodied in this thesis is the result of original research, is free of plagiarised materials, and has not been submitted for a higher degree to any other University or Institution.

7/11/2024

Date

WANG SU

Supervisor Declaration Statement

I have reviewed the content and presentation style of this thesis and declare it is free of plagiarism and of sufficient grammatical clarity to be examined. To the best of my knowledge, the research and writing are those of the candidate except as acknowledged in the Author Attribution Statement. I confirm that the investigations were conducted in accord with the ethics policies and integrity standards of Nanyang Technological University and that the research data are presented honestly and without prejudice.

7/11/2019

Date

Tan Kang Hai

Authorship Attribution Statement

This thesis contains material from 2 papers published in the following peer-reviewed journals where I was the first author.

Chapter 3, Section 6.2.1, 6.2.2, and 6.3.1.1 in Chapter 5, Section 5.2.1, 5.2.2 and 5.3.2 in Chapter 6 were published as: Su Wang, Kang Hai Tan, Evaluation on the performance of lightweight cementitious composite enhanced by carbon nanofibers, Proceedings of the fib Symposium 2019: Concrete - Innovations in Materials, Design and Structures

- Prof Tan Kang Hai provided the initial project direction
- All the testing samples were prepared by me
- All the tests were conducted by me in the Protective Engineering Lab and Construction Technology Lab at the School of Civil and Environmental Engineering
- Prof Tan Kang Hai provided guidance in the interpretation of the results
- I wrote the drafts of the manuscript. The manuscript was revised by Prof Tan Kang Hai

Section 6.3.3.2 to 6.3.3.3 in Chapter 5 and Appendix A were published as: Su Wang, Shao-Bo Kang, Kang Hai Tan, Evaluation of bond-slip behaviour of embedded

rebars through control field equations, Magazine of Concrete Research 71 (17) 907-919. DOI: 10.1680/jmacr.17.00509.

The contributions of the co-authors are as follows:

- Prof Tan Kang Hai and Prof Kang Shao Bo provided the initial project direction
- All the analysis, calculation and programming were conducted by me
- Prof Tan Kang Hai and Prof Kang Shao Bo provided guidance in the interpretation of the results
- I wrote the drafts of the manuscript. The manuscript was revised by Prof Tan Kang Hai and Prof Kang Shao Bo.

7/11/2019

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ABSTRACT

As a type of lightweight concrete, foam concrete is traditionally applied in building industry for its thermal and acoustic insulation properties. However, in recent years, there is a surge in interest in potential applications of foam concrete as a structural component due to its low self-weight, saving in raw materials and sustainability. The main challenge for foam concrete is to have high-performance pore walls to provide acceptable mechanical properties and other engineering properties under reduced density. In this study, a new type of structural foam concrete termed as carbon nanofibers enhanced lightweight cementitious composite (CNF-LCC) was developed based on nano-engineered ultra-high performance concrete technology. The potential application of CNF-LCC on structures was investigated by a series of comprehensive experimental programmes from the material level to the structural level and in-depth analysis of the testing results was discussed.

CNF-LCC was produced by using carbon nanofibers (CNFs) enhanced ultra-high performance concrete (ceUHPC) as a base mix and then blending with homogeneous micro-foam bubbles to achieve a density of $1500 \pm 50 \text{ kg/m}^3$. The basic mechanical properties of CNF-LCC were measured in accordance with standard codes (European or American codes) and they are superior to conventional foam concrete. CNFs showed effective improvement on the mechanical properties especially flexural strength and toughness. The thermal properties of CNF-LCC under high temperature

indicated its reasonably good thermal insulation properties and low thermal expansion for fire resistance. The phase transformations of CNF-LCC under high temperatures were characterised to elaborate on the experimental results and CNFs could reduce thermal shrinkage without degrading thermal insulation properties. As a critical performance for structural building material, the long-term properties of CNF-LCC were evaluated by testing the durability, shrinkage and creep. CNF-LCC presented excellent long-term performance compared to conventional normal and lightweight concrete due to the optimised UHPC base mix and modified characteristics of pore structure by CNFs. Prior to the structural tests, the bond performance between CNF-LCC and steel reinforcement was studied by pullout tests of short and long embedment length. The bond strength of CNF-LCC exceeded the traditional foam concrete and was comparable with normal concrete (NWC) and lightweight aggregate concrete (LWAC) due to the improved mechanical properties. A new analytical model was proposed to accurately predict the bond-slip performance of concrete including CNF-LCC. Finally, the flexural performance of reinforced CNF-LCC beams was investigated and the experimental behaviour surpassed conventional foam concrete and was comparable with NWC and LWAC. CNFs presented comprehensive improvement on flexural performance, especially beam ductility. Recommendations for the design and analysis of reinforced CNF-LCC beams were provided and an analytical model was proposed to predict the load-deflection relationship.

In general, CNF-LCC based on nano-engineered UHPC technology presented outstanding material properties and structural performance compared to conventional foam concrete. The experimental results not only established the database in the use of CNF-LCC but also provide more test results for foam concrete. At the same time, the effect of CNFs was comprehensively investigated from the nanostructure of material to the macrostructure of elements, which was absent in previous research work. Furthermore, the related recommendations and prediction models for CNF-LCC were proposed to guide the design and analysis of reinforced CNF-LCC members in the future. CNF-LCC provided a new solution for structural lightweight concrete.

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LIST OF SYMBOLS

A, B	constants in hyperbolic expression for creep development
A_s, P_s	cross-sectional area and perimeter of reinforcement respectively
A_{st}, A_{sc}	area of tension and compression reinforcement respectively
a	length of shear span of beam
b	width of beam cross section
c_1	compression depth
c_2	depth of confined concrete zone
c_3	depth of unconfined concrete zone
c_p	specific heat of concrete
c_v	concrete cover
d_b	diameter of reinforcement
d	effective depth of the beam cross section
d'	distance between the centroid of the compression bar and extreme compression fiber of the section
d_s	diameter of links
e	depth of penetration
E	modulus of elasticity of reinforcement
E_c	elastic modulus of NWC
$E_{c,eff}$	effective elastic modulus of concrete
E_{lc}	elastic modulus of LWAC

E_h hardening modulus of reinforcement
 F load applied to embedded reinforcement
 F_u ultimate force of reinforcement
 F_r fracture force of reinforcement
 F_y yield force of reinforcement
 f_{cm} 28-day compressive strength of concrete
 f_c' cylinder compressive strength of concrete
 f_{ck} characteristic cylinder compressive strength of concrete
 f_r flexural tensile strength of NWC
 f_{lr} flexural tensile strength of LWAC
 f_t direct tensile strength of NWC
 f_{lt} direct tensile strength of LWAC
 f_{yc} yielding strength of compression reinforcement
 f_{ys} yielding strength of transverse reinforcement
 f_{yt} yielding strength of tensile reinforcement
 h total member depth
 I_g moment of inertia for the gross section
 h_w hydraulic head of water
 K coefficient of permeability
 L length of total span of beam
 l embedment length of reinforcement

l_d development length of reinforcement

l_e, l_y length of elastic and post-yield state of reinforcement respectively

l_{e1}, l_{e2} length of first and second segment of elastic reinforcement respectively

l_r required embedment length of reinforcement

l_1 limited length of reinforcement

$M_{cr}, M_y, M_u, M_{sp}, M_f$ flexural moment at cracking, yielding, ultimate, spalling and failure respectively

$M_{c,ACI}, M_{c,EC2}$ and $M_{c,CEB}$ predicted cracking moment from ACI, EC 2 and CEB-FIB codes respectively

$M'_{c,ACI}, M'_{c,EC2}$, and $M'_{c,CEB}$ predicted cracking moment with safety factor from ACI, EC 2 and CEB-FIB codes respectively

$M_{c,exp}$ experimental cracking moment

$M_{u,exp}$ experimental ultimate moment

$M_{u,ACI}, M_{u,EC2}$ and $M_{u,CEB}$ predicted ultimate moment from ACI, EC 2 and CEB-FIB codes respectively

RH relatively humidity

s slip of reinforcement

s' first derivative of slip of reinforcement

s'_f first derivative of slip at free end of reinforcement

s_e, s_h slips at both ends of steel segment

s_f slip at free end of reinforcement

s_y slip at yield point of reinforcement

s_1, s_2, s_3 slip of reinforcement to define Eligehausen's model

t_p test duration of water penetration depth

t loaded age in creep test

T temperature

v fraction of concrete volume occupied by pores

w/c water to cement ratio

ω_{exp} experimental maximum crack width

$\omega_{ACI}, \omega_{EC2}$ and ω_{CEB} predicted maximum crack width from ACI, EC 2 and CEB-FIB codes respectively

x_f, x coordinate of free end and calculated point along embedded reinforcement

y_b distance from the neutral axis to the extreme tension fiber of the section

Δl length of steel segment

α parameter to define Eligehausen's model

α_p thermal diffusivity of concrete

$\delta_{ACI}, \delta_{EC2}$ and δ_{CEB} predicted deflection at service load from ACI, EC 2 and CEB-FIB codes respectively

$\delta'_{ACI}, \delta'_{EC2}$ and δ'_{CEB} predicted deflection at service load considering creep from ACI, EC 2 and CEB-FIB codes respectively

δ_{exp} experimental deflection at service load

δ_y, δ_f deflection at yielding and failure point

ε_c compressive strain of concrete

ε_{cu} maximum strain of confined concrete

ε_{cm} concrete strain at compressive strength

$\varepsilon_e, \varepsilon_h$ strains at both ends of steel segment

ε_{e1} tensile strain at $x = l_{e1}$ of reinforcement

ε_f tensile strain at free end of reinforcement

ε_s tensile strain of reinforcement

ε_{sc} strain of the compression reinforcement

ε_{st} strain of the tensile reinforcement

ε_y yield strain of reinforcement

ε_{yc} yielding strain of compression reinforcement

ε_{yt} yielding strain of tensile reinforcement

ρ tension reinforcement ratio

ρ' compression reinforcement ratio

ρ_{bal} balance reinforcement ratio

ρ_d unit weight of concrete

ρ_s transverse reinforcement ratio

σ_c compressive stress of concrete

σ_e, σ_h stresses at both ends of steel segment

σ_{e1} tensile stress at $x = l_{e1}$ of reinforcement

σ_l tensile stress at loaded end of reinforcement

σ_s tensile stress of reinforcement

σ_u ultimate strength of reinforcement

σ_y yield strength of reinforcement

σ_1 limited stress of reinforcement

λ thermal conductivity of concrete

τ bond stress of reinforcement

τ_e, τ_y bond stress at elastic and post-yield stage of reinforcement respectively

τ_m, τ_f maximum and frictional bond stress respectively

$\varphi_{cr}, \varphi_y, \varphi_u, \varphi_{sp}, \varphi_f$ moment curvature at cracking, yielding, ultimate, spalling and failure respectively.

LIST OF ACRONYMS

AAC	autoclaved aerated concrete
ceUHPC	carbon nanofibers enhanced ultra-high performance concrete
CNFs	carbon nanofibers
CNTs	carbon nanotubes
CNF-LCC	carbon nanofibers enhanced lightweight cementitious composite
C-S-H	calcium silicate hydrate
CVD	chemical vapor deposition
Demec	demountable mechanical
HPLC	high performance lightweight concrete
HSC	high strength concrete
ITZ	interfacial transition zone
LCC	lightweight cementitious composite
LVDT	linear variable differential transducer
LWAC	lightweight aggregate concrete
LWC	lightweight concrete
LWSCC	lightweight self-compacting concrete
MDSC	modulated Differential Scanning Calorimetry
MIP	mercury intrusion porosimetry
NWC	normal weight concrete
PAC	Polystyrene aggregate concrete

SEM	scanning electron microscope
SLWC	structural lightweight concrete
TEM	transmission electron microscope
TGA	thermogravimetric analysis
UHPC	ultra-high performance concrete
XRD	X-ray Diffraction

CHAPTER 1 INTRODUCTION

1.1 Research Background

Based on density, concrete can be classified as normal weight concrete (NWC) or lightweight concrete (LWC). Generally, the density of NWC hovers around 2200 to 2600 kg/m³ while LWC ranges from 300 kg/m³ to 1900 kg/m³ (Neville, 1995).

The benefits of using LWC are the reduction of construction costs, improved functionality, or a combination of both (ACI Committee 213, 2003). The economic effect is mainly attributed to the lower density of LWC which reduces the dead load. The reduced dead load results in smaller sizes of supporting members and lower amount of reinforcement, which create major cost saving in construction. Meanwhile, the reduced weight of the overall structure means lower foundation loads making smaller footings, fewer piles, and smaller pile caps possible. Moreover, lightweight structural members directly save the cost of transportation in precast construction. The increased functionality also makes LWC attractive. For a given bearing capacity of soil, a larger volume of structures can be built with the same foundation works due to the use of LWC. In precast construction, longer and larger members can be produced without increasing overall mass. This would result in lighter structural elements in a system that is easier to transport, lift or erect. Another important property of LWC is the lower thermal conductivity due to the porous matrix. This is

beneficial to substantial energy saving and fire resistance. In addition, the reduced dead load of LWC decreases the inertial shear forces during an earthquake.

The principle of producing LWC is to replace some of the solid materials in the matrix by air voids. If air voids are introduced into aggregates, i.e. lightweight aggregates, the final product is known as lightweight aggregate concrete (LWAC). If parts of cement paste are replaced by air voids, the overall matrix is cellular concrete. According to ACI 213R-03 (2003), LWAC is defined as structural lightweight concrete (SLWC) if it can achieve a minimum 28-day compressive strength of 17 MPa. Unlike LWAC, cellular concrete is mainly applied for non-structural purposes due to its lower mechanical properties. Based on the pore-formation methods in cement paste, cellular concrete can be divided into aerated concrete and foam concrete. Aerated concrete is produced by mixing an aerated agent with cement mortar, resulting in a chemical reaction during which hydrogen is produced and a porous structure is formed after the gas has escaped from the matrix. However, the size of air pores may be large and uneven due to uncontrollable chemical reactions (Narayanan and Ramamurthy, 2000). Therefore, aerated concrete is usually cured under autoclaved condition to increase mechanical properties, also known as autoclaved aerated concrete (AAC). This process not only consumes high energy but also places limitations on the dimensions of AAC components (Schauerte and Trettin, 2011, Jones and McCarthy, 2005a).

However, foam concrete is an alternative material to overcome the disadvantages of AAC (Cox and van Dijk, 2002, Brady et al., 2001). The air pore in foam concrete is introduced through a mechanical approach either by pre-foaming method or mixed foaming method (Ramamurthy et al., 2009). The mechanically generated air pore is much more economical and controllable because no chemical reaction is involved (Narayanan and Ramamurthy, 2000) and it does not require autoclave process. Conventional foam concrete is designed with low mechanical properties for non-structural applications such as partition walls, void fill, and insulation (Ramamurthy et al., 2009, Jones and McCarthy, 2005). It is a challenge for foam concrete to achieve reliable mechanical properties at a relatively lower density. In addition, limited literature shows that foam concrete has higher shrinkage, extremely lower bond strength and higher permeability than NWC and LWAC. Furthermore, as one of the indispensable properties of structural materials, creep of foam concrete has not been studied until now. The studies on reinforced foam concrete elements are also scarce but all of them show degraded performance due to deficient engineering properties of foam concrete. However, there is a growing interest in structural applications of foam concrete in view of its lightweight, material/energy saving during production, self-compacting/leveling, excellent thermal and acoustic insulation (sustainability), and fire resistance (Jones and McCarthy, 2005a, Cox and van Dijk, 2002, Ramamurthy et al., 2009, Narayanan and Ramamurthy, 2000).

1.2 Scope and Objective of This Research

In this research, the methods of ultra-high performance concrete (UHPC) and nano-technology based on carbon nanofibers (CNFs) were combined to effectively improve foam concrete. CNFs enhanced UHPC (ceUHPC) was used as a base mix and then blended with homogeneous micro-foam to produce a lightweight cementitious composite (CNF-LCC) with $1500 \pm 50 \text{ kg/m}^3$ density for structural applications. Comprehensive experimental programmes from material to structural level were conducted to exhibit the potential of structural applications of CNF-LCC. An in-depth analysis of the test results was presented. The performance of CNF-LCC was evaluated by comparing with traditional foam concrete, NWC and LWAC from a wide range and examined by the related standard codes. The influence of CNFs on the microstructure and performance was studied by comparing CNF-LCC with the control samples LCC (matrix without CNFs). The study could be divided into four research areas and each of them was self-contained by its own right and yet they were closely related. These four research areas were: (1) mix design of CNF-LCC; (2) short-term properties of CNF-LCC; (3) long-term properties of CNF-LCC; and (4) flexural performance of reinforced CNF-LCC beams.

The objective of the first research area was to obtain the mix design of CNF-LCC by blending nano-engineered UHPC base mix with homogeneous micro-foam. The UHPC base mix was modified by optimal packing density of solid materials and

enhancement of CNFs. The fresh properties of CNF-LCC mix were evaluated by the flowability and the characteristics of micro-foam structures.

The objective of the second research area was to investigate the short-term properties of CNF-LCC including mechanical properties, thermal properties, and bond behaviour with reinforcement. Standard testing methods were used to determine the basic mechanical properties of CNF-LCC including compressive strength, flexural strength, toughness, elastic modulus and compressive stress-strain curve. The thermal properties consisted of thermal diffusivity, specific heat, thermal conductivity and thermal strain. The effects of temperature, foam bubbles and CNFs on the thermal properties were investigated according to phase transformations. The bond behaviour between steel reinforcement and CNF-LCC was obtained by conducting pull-out tests. The major parameters of interest are the embedment length, bar diameter and CNFs. An analytical model was proposed to predict the bond behaviour of CNF-LCC.

The third research area introduced the long-term mechanical properties of CNF-LCC including durability (permeability), one-year shrinkage and creep. The pore size distribution was determined and the effect of CNFs was presented to explain the influence on these properties. The experimental results were compared with NWC or LWAC to evaluate the time-dependent properties of CNF-LCC. Finally, an analytical model was proposed to describe the creep development of CNF-LCC.

The objective of the fourth and last research area was to study the performance of reinforced CNF-LCC beams under flexural load. The parameters studied include the existence of CNFs, tension reinforcement ratio, compression reinforcement ratio, and transverse reinforcement ratio. The complete flexural response of reinforced CNF-LCC beams was investigated and examined by EC 2 requirements and compared with corresponding NWC, LWAC and foam concrete. The applicability of different standard codes to predict the flexural response of CNF-LCC beams was evaluated and an analytical model was applied to predict the complete load-deflection relationship of reinforced CNF-LCC beams.

1.3 Layout of the Thesis

This thesis is divided into eight chapters. The contents of the following chapters are briefly described as follows.

Chapter Two presents an overview of previous research work that is relevant to carbon nanofibers enhanced lightweight cementitious composite (CNF-LCC). Since CNF-LCC is a completely innovative material, this review focuses on the material level consisting of foam concrete and carbon nanofibers enhanced concrete.

Chapter Three introduces the optimisation concept and the mix design of CNF-LCC. Afterwards, the mechanical properties of CNF-LCC tested by standard methods along with the analysis of experimental results are reported.

Chapter Four reports the thermal properties of CNF-LCC under high temperature including thermal diffusivity, specific heat, and thermal expansion. The phase transformations of samples under high temperature are studied to explain the phenomenon.

Chapter Five introduces the long-term properties of CNF-LCC including durability (permeability) and one-year shrinkage and creep. The pore size distribution is determined and the link between CNFs and mechanical properties is explained. An analytical model is proposed to describe the creep development of CNF-LCC.

Chapter Six presents the bond behaviour of deformed steel reinforcement embedded in CNF-LCC at elastic and post-yield stress state by conducting pullout tests with short and long embedment length of reinforcement. An analytical model based on field control equation is proposed to predict the experimental results and its accuracy is validated.

Chapter Seven presents the performance of reinforced CNF-LCC beams under flexural load to explore structural applications. Four parameters are considered and the complete flexural response of reinforced CNF-LCC beams is investigated at both serviceability and ultimate limit states. The load-deflection relationship of reinforced CNF-LCC beams is predicted by a proposed model.

Chapter Eight concludes the research work and gives recommendations for the future work.

CHAPTER 2 LITERATURE REVIEW

2.1 Overview

Carbon nanofibers (CNFs), carbon nanofiber lightweight cementitious composites (CNF-LCC) and carbon nanofiber enhanced foam concrete are innovative construction materials and, as such, few studies are reported in the literature. In this section, pioneering studies on these materials that have been undertaken over the past few decades are reported and discussed. In particular, the constituent materials, engineering properties and structural applications and of foam concrete are reviewed. In the section of CNFs enhanced concrete, the growth mechanism and morphology of CNFs, dispersion of CNFs in water and their effect on concrete are introduced. These basic principles pave the way for the development of CNF-LCC, undertaken in this study and reported in the following chapters.

2.2 Foam Concrete

2.2.1 Constituent materials

2.2.1.1 Base mix

Cement is the most commonly used binder material in foam concrete. In addition to Ordinary Portland cement, rapid hardening Portland cement, calcium sulfoaluminate cement and high alumina cement are also used in foam concrete to decrease the setting time and increase the early-age mechanical properties (Jones and McCarthy,

2005a, Kearsley and Wainwright, 2001a, Turner, 2001). Supplementary cementitious materials such as fly ash, ground granulated blast slag and silica fume can be used to replace a certain percentage of cement to reduce hydration heat and increase the short- or long-term strength of foam concrete. (Durack and Weiqing, 1998, Norlia et al., 2013, Fujiwara et al., 1995). Only fine aggregates are used in foam concrete because coarse aggregates will destroy the air bubbles during the mixing process. Various fine aggregates were used including fly ash (Durack and Weiqing, 1998), lime (De Rose and Morris, 1999), crushed concrete (Aldridge and Ansell, 2001), recycled glass, foundry sand and quarry finer (Jones et al., 2005), expanded polystyrene and Lytag fines (Deijk, 1991), and lightweight aggregates (Regan and Arasteh, 1990). Water content is critical in foam concrete as it controls the workability of matrix and ensures the integrity of foam bubbles. Low water content makes the mix too stiff resulting in bubbles breaking while high water content causes the mix to be too thin to hold the bubbles (Nambiar and Ramamurthy, 2006). Superplasticiser can modify the flowability of base mix but the usage of it may result in instability of foam and thus it is not usually used (Jones and McCarthy, 2006). Inclusion of micro-fibers such as polypropylene fibers was also reported to improve brittle behaviour of foam concrete (Bing et al., 2011).

2.2.1.2 Foam agent

In foam concrete, the air bubbles are created by physical approaches including *pre-foaming method* and *mixed foaming method*. In the former, stable foam is produced in advance by mixing foam agent solution with compressed air and then blending it with the base concrete mix. The mixed foaming method is to directly mix the foam agent solution with other concrete ingredients and the foam is generated from mechanical mixing force but the sizes of the foam bubbles are heterogeneous and uncontrollable. Hence, pre-foaming method is preferred because individual production of foam ensures its stability and uniformity. Commonly utilized foam agents are synthetic and protein-based although researchers also used detergents, resin soap, glue resins or saponin (De Rose and Morris, 1999, Deijk, 1991). The foam bubbles produced by protein-based foam agents are stronger and closed-cell resulting in a higher amount of included air (Beningfield et al., 2005). The synthetic agents can generate large expansion, which is appropriate for producing low-density foam concrete (Amran et al., 2015). Jones and McCarthy (2005) reported that the category of foam agent significantly affected the compressive strength of foam concrete. The produced foam should be strong and stable in order to withstand the pressure from the cement paste/mortar until the cement reaches the initial set and forms a strong concrete skeleton around the foam bubbles (Koudriashoff, 1949). The stability of foam bubbles is controlled by the foam agents concentration, viscosity of liquid phase,

surface effects and disjoining pressure between adjacent interfaces (Pugh, 1996, Tan et al., 2005b).

2.2.2 Mechanical properties of foam concrete

2.2.2.1 Compressive strength

The compressive strength of foam concrete reduces exponentially with decreasing density (Kearsley, 2006). When comparing the compressive strength of foam concrete with that of LWAC and AAC at similar density level, foam concrete usually has lower compressive strength (Jones and McCarthy, 2005). LWAC is mainly applied to structural components if it has a minimum of 17 MPa compressive strength. Autoclaved aerated concrete is cured by high pressure and temperature which accelerates the hydration process to increase compressive strength. Jones and McCarthy (2005a) reported a research programme studying the potential of foam concrete as a structural material. In their work, engineering properties of foam concrete with density varying from 1400 to 1800 kg/m³ were measured and the reinforced foam concrete beams were produced to study the flexural performance. The test results showed that foam concrete was indeed viable for structural usage. However, they indicated that the 28-day compressive strength of foam concrete could only exceed 25 MPa (minimum strength for structural use) at the density of 1800 kg/m³ (Jones and McCarthy, 2005a). It correlated well with the review about the strength and density of foam concrete from Ramamurthy et al. (2009). A summary of

material compositions and properties of foam concrete is included in Table 2.1. It shows similar conclusions with previous work that most foam concrete had compressive strength lower than 25 MPa, which limited them to non-structural applications. Bing et al. (2011) reported a type of high strength foam concrete for structural applications and the 28-day compressive strength reached 46 MPa at the density of 1500 kg/m³ based on 100 mm cubes. The high compressive strength was attributed to low water-cement ratio, inclusion of fly ash and silica fume, absence of sands, and introducing polypropylene fibers. Such material composition significantly increased the compressive strength of foam concrete but the risk of inadequate elastic modulus, large shrinkage and creep deformation is high due to the absence of sands.

Table 2.1 A summary of material composition, density and 28-day compressive strength of foam concrete

Researchers	Solid material compositions	Water/binder ratio	Density (kg/m ³)	28-day compressive strength (MPa)
McCormick (1967)	Cement	0.35 ~ 0.57	800~1800	1.8~17.6
Tam et al. (1987)	Cement	0.60 ~ 0.80	1300~1900	1.8~16.7
Kearsley and Booyens (1998)	Cement and fly ash		1000~1500	2.8~19.9
Aldridge (2000)	Cement and sand		400~1600	0.5~10
Kearsley and Wainwright (2001a)	Cement and fly ash		1000~1500	2~18
Brady et al. (2001)	Cement and sand	0.30 ~ 0.40	1800	28
Tikalsky et al. (2004)	Cement	0.40 ~ 0.45	1320~1500	0.23~1.1
Jones and McCarthy (2005a)	Cement and sand	0.26 ~ 0.50	1400 ~ 1800	10 ~ 26
	Cement and fly ash	0.26 ~ 0.50	1400 ~ 1800	20 ~ 43
Kishore (2007)	Cement and fly ash	0.35 ~ 0.50	800 ~ 1400	2.5 ~ 12.0
	Cement and sand	0.40 ~ 0.55	1200 ~ 1800	6.5 ~ 25.0
	Cement, sand and fly ash	0.40 ~ 0.55	1200 ~ 1800	6.5 ~ 25.0

Bing et al. (2011)	Cement, fly ash and PP fibers	0.30 ~ 0.60	800 ~ 1500	8 ~ 37
	Cement, fly ash, silica fume and PP fibers	0.30 ~ 0.60	800 ~ 1500	12 ~ 46
Roslan et al. (2013)	Cement, fly ash, lime and PP fibers	0.45 ~ 1.00	1400	5.4 ~ 13.2
Richard and Ramli (2013)	Cement and fly ash	0.30	1500 ~ 1800	3.9 ~ 10.5
Norlia et al. (2013)	Cement, sand and sludge aggregate	0.50	1837	25

2.2.2.2 Tensile strength

Tensile strength of foam concrete is lower than that of NWC and LWAC (Ramamurthy et al., 2009). Byun et al. (1998) indicated that the ratio of tensile strength to compressive strength of foam concrete was around 0.2 to 0.4, which was higher than NWC with a ratio of splitting tensile strength to compressive strength from 0.08 to 0.11. However, Jones and McCarthy (2005a) reported that the splitting tensile to compressive strength of foam concrete was lower than the calculated results of NWC and LWAC. The flexural tensile strength to compressive strength ratio of foam concrete ranges from 6% to 10%, lower than NWC and LWAC with comparable compressive strength (Deijk, 1991). When the density of foam concrete was less than 300 kg/m^3 , the ratio of flexural to compressive strength was nearly zero. The mix design with sands or other mineral additives as fine aggregates has greater tensile strength than that with fly ash due to increased shear capacity between the sand particles and the cement paste (Valore, 1954a, Jones, 2001). Furthermore, it was reported that adding flexible polypropylene fibers in foam concrete can significantly improve the tensile strength, flexural toughness and post cracking behaviour (Jones and McCarthy, 2005, Bing et al., 2011, Zollo and Hays, 1998).

2.2.2.3 Modulus of elasticity

The elastic modulus of foam concrete is significantly lower than that of NWC and LWAC with an equivalent compressive strength (Ramamurthy et al., 2009). For the

density of foam concrete between 500 and 1500 kg/m³, the experimental values of elastic modulus ranged from 1 to 8 GPa (Jones and McCarthy, 2005, Brady et al., 2001). The elastic modulus of foam concrete was only a quarter of that of NWC with an equivalent compressive strength (Jones and McCarthy, 2005). Similar to tensile strength, foam concrete with sand as fine aggregate exhibits higher modulus of elasticity than that with coarse fly ash (Jones, 2001). The lower tensile strength and lower modulus of elasticity limit applications of foam concrete in flexural elements due to premature cracking and larger deflection. It was found that adding polypropylene fibers could increase the elastic modulus of foam concrete from 2 to 4 times (Jones and McCarthy, 2005).

2.2.3 Shrinkage and creep

Shrinkage is considered as one of the drawbacks of foamed concrete since it ranges from 4 to 10 times higher than that of NWC due to the absence of coarse aggregates, higher water content and mineral admixture in foamed concrete (Jones et al., 2003, Roslan et al., 2013, McGovern, 2000). The shrinkage of foam concrete decreases with reducing density (increasing foam volume) due to lower cement paste content (Jones et al., 2003, Tada and Nakano, 1983, Schubert, 1983, Nambiar and Ramamurthy, 2009, Nmai, 1997). In addition, higher foam volume increases the size of foam bubbles resulting in lower shrinkage because water lost from relatively larger pores does not generate significant shrinkage (Nambiar and Ramamurthy, 2009). To reduce

the negative effect of cement paste on shrinkage, partial replacement of cement by other supplementary materials like fly ash, silica fume, and lime was tried (Chindaprasirt et al., 2008). Meanwhile, increasing the content of sands or lightweight aggregates can effectively reduce shrinkage of foam concrete because of their higher shrinkage restraining capacity (Jones et al., 2003, Weigler and Karl, 1980, Regan and Arasteh, 1990). It was also reported that autoclaving curing process can effectively reduce shrinkage of foam concrete because of the change of mineralogical compositions, and it can increase the strength simultaneously (Valore, 1954a). Furthermore, it is recommended to reduce water-binder ratio and use appropriate category and volume of foam agent to reduce shrinkage (Ramamurthy et al., 2009, Mellin, 1999). Research on creep of foam concrete is very scarce because it is mainly used for non-structural applications.

2.2.4 Durability

2.2.4.1 Permeation characteristics

Durability of foam concrete is closely connected with its permeation characteristics. Jones and McCarthy (2005) summarised that oxygen, air and water vapor permeability of foam concrete are greater than those of normal concrete and increases with a reduction of density because artificial air voids make significant contribution to permeation (Kearsley and Wainwright, 2001b). However, water absorption and sorptivity of foam concrete show the opposite trend and they reduce with an increase

in foam amount due to lower capillary porosity. These two permeation properties are mainly controlled by capillary pores in cement paste but not the foam pores (Nambiar and Ramamurthy, 2007).

2.2.4.2 Freeze-thaw resistance

Foam concrete with a density between 800 and 1400 kg/m³ has shown outstanding freeze-thaw resistance (Jones and McCarthy, 2005). This is because the foam air bubbles act as “safety valves” drawing water from the cement paste and serving as minute and discrete reservoirs. In this way, it can release the pressure in the capillary pores of cement paste during freezing.

2.2.4.3 Sulfate resistance

Foam concrete is usually used as a filler material in foundations where sulfate may exist in surrounding soil and groundwater. An experiment to investigate the sulfate resistance of foam concrete was conducted by Jones and McCarthy (2005) and it was shown that foam concrete has great resistance to sulfate attack after one year. This may be attributed to the air voids in foam concrete providing additional spaces for volumetric expansion of sulfate reaction, hence, reducing cracks caused by pressure.

2.2.4.4 Corrosion of reinforcement

Penetration of chlorides and carbonation of concrete are the main factors causing corrosion of reinforcement. Kearsley and Booyens (1998) conducted accelerated

chloride ingress tests for both the foam concrete and the normal concrete with equivalent compressive strength. The foam concrete showed identical performance to the normal concrete from the test results. An accelerated carbonation test by placing the foam concrete specimen in an enriched CO₂ environment indicated that the foam concrete had significantly higher carbonation compared to normal concrete for the same period and higher foam amount increased the carbonation depths (Jones and McCarthy, 2005). This may be explained by the high gas diffusivity of foam concrete which increases the rate and depth of carbonation penetration (Jones and McCarthy, 2005).

2.2.5 Thermal properties

The cellular microstructure gives foam concrete excellent thermal insulation property because the introduced air bubbles have almost negligible thermal conductivity (i.e. 0.026 W/mK). The literature reports on thermal conductivity of different foam concrete mix designs at room temperature; however, no studies have investigated thermal conductivity at high temperatures. Table 2.2 summarises the studies on thermal conductivity, density and solid materials used in the mix designs of foam concrete at ambient temperature. It was found that thermal conductivity of foam concrete ranges from 0.06 to 0.7 W/mK for density varying from 150 to 1600 kg/m³, while NWC has thermal conductivity of 1.6 W/mK at 2200 kg/m³ (Amran et al., 2015). The thermal conductivity of foam concrete decreases with reducing density

due to an increased volume of air (Ramamurthy et al., 2009). Addition of mineral admixtures such as fly ash or pulverized fuel ash effectively reduces thermal conductivity because the lower density and cenospheric particle morphology of particles delay heat transfer through the material (Mydin et al., 2012, Giannakou and Jones, 2002, Jones and McCarthy, 2006). Meanwhile, adding fly ash or nanomaterials such as carbon nanotubes promote uniform pore size distribution and reduce pore size by preventing air bubbles from merging with one another, which subsequently reduces the thermal conductivity of foam concrete (Mydin et al., 2012, Yakovlev et al., 2006). It was also observed that the inclusion of lightweight aggregates in foam concrete is beneficial in reducing thermal conductivity (Zhang et al., 2014, Aldridge and Ansell, 2001). In addition to thermal insulation, fire resistance is another important functional property of foam concrete. Available research works have shown acceptable or even superior fire resistance of foam concrete when compared to NWC but they were largely qualitative (Vilches et al., 2012, Sach and Seifert, 1999, Kearsley and Mostert, 2005, Valore, 1954b, Jones and McCarthy, 2005a, Aldridge, 2005). The knowledge of thermal properties at elevated temperature is critical for evaluating the fire response of concrete structures (Kodur and Khaliq, 2010). However, there is a lack of quantitative experimental work in this area for foam concrete.

Table 2.2 A review of solid materials used, density and thermal conductivity of foam concrete

Researchers	Solid material composition	Density (kg/m ³)	Thermal conductivity (W/mK)
British Cement Association (1994)	Cement, sands	600 ~ 1600	0.1 ~ 0.7
Aldridge and Ansell (2001)	Cement, sands, lightweight aggregates	1000	1/6 of typical cement-sands mortar
Giannakou and Jones (2002)	Cement, sands	1000 ~ 1400	0.36 ~ 0.55
	Cement, fly ash, sand	1000 ~ 1400	12% ~ 38 lower than those only have cement
Proshin et al. (2005)	Cement, sands, polystyrene granules	200 ~ 650	0.06 ~ 0.16
Jones and McCarthy (2006)	Cement, sands	1000 ~ 1200	0.23 ~ 0.42
Yakovlev et al. (2006)	Cement	330	0.07
	Cement, carbon nanotubes	309	0.056
Mydin (2011)	Cement, sands	650 ~ 1200	0.23 ~ 0.39
Kim et al. (2012)	Cement, fly ash, lightweight aggregates	1200 ~ 1500	0.37 ~ 0.54
Vilches et al. (2012)	Cement	150 ~ 500	0.08 ~ 0.19
Mydin et al. (2012)	Cement, sands	600 ~ 1400	0.19 ~ 0.59
	Cement, sands, fly ash	600 ~ 1400	0.16 ~ 0.61
	Cement, sands, lime	600 ~ 1400	0.16 ~ 0.59
	Cement, sands, Polypropylene fiber	600 ~ 1400	0.18 ~ 0.60
Zhang et al. (2014)	Cement, lightweight aggregates	800 ~ 1050	0.24 ~ 0.37
Zhang et al. (2014)	Cement, sands	400 ~ 1600	0.10 ~ 0.64
Silva et al. (2015)	Cement	130 ~ 1000	0.046 ~ 0.35
Zhang et al. (2015)	Fly ash, slag	585 ~ 1370	0.15 ~ 0.48

Othuman and Wang (2011) proposed analytical models to predict thermal conductivity and specific heat of foam concrete with a density of 650 kg/m^3 and 1000 kg/m^3 from 20 to 1000°C . They conducted transient heating tests on foam concrete slabs and compared the experimental results with the analytical results from a one-dimensional heat transfer program to validate the predicted thermal properties. They used hot guarded plate (HGP) instrument to directly measure the thermal conductivity of foam concrete up to 250°C (maximum operating temperature of the instrument). However, up to now, there was no experimental result for foam concrete for thermal conductivity measurements from 250 to 1000°C and specific heat capacity from 20 to 1000°C .

In addition to thermal conductivity and specific heat capacity, thermal expansion is another important thermal property of concrete (Uygunoğlu and Topçu, 2009). Concrete with high thermal expansion may suffer from serious local damage caused by thermal stress because local expansion is restrained by surrounding cooler parts (Neville, 1995). Meanwhile, high thermal expansion of concrete increases the tendency of buckling when two faces of a concrete element are exposed to different temperatures (Neville, 1995). However, research on thermal expansion of foam concrete is limited. Only Sach and Seifert (1999) reported that foam concrete showed excessive shrinkage due to high evaporation rates when exposed to elevated temperature.

2.2.6 Behaviour of reinforced foam concrete beams

Research on flexural performance of reinforced foam concrete is scarce because such material is mainly used for non-structural applications. Jones and McCarthy (2005a) conducted pilot tests to compare the flexural behaviour of 1400 and 1600 kg/m³ foam concrete beams with NWC beams of similar compressive strength (25 MPa). Both foam concrete and NWC beams failed in tension after yielding of steel bars and their ultimate loads were close since they had similar compressive strength. However, the cracking load of foam concrete beams was significantly lower than their normal concrete counterpart, which was attributed to lower flexural tensile strength of foam concrete. Meanwhile, the load-deflection relationship indicated that the stiffness of foam concrete beams was considerably smaller than NWC beams due to its lower elastic modulus and more brittle behaviour. This study gave a simple comparison of flexural performance between foam concrete and NWC beams but detailed flexural responses at service load and post-peak period were not discussed. Lim (2007) conducted a comprehensive test programme on reinforced foam concrete beams with 20 and 35 MPa compressive strength to study the complete flexural response. Similar to the conclusions in Jones and McCarthy (2005a), Lim (2007) found that foam concrete beams showed tension failure mode and similar ultimate strength compared with NWC and LWAC of comparable strength. However, the maximum crack width in foam concrete beams at service load was much greater than that of NWC and

LWAC beams and exceeded the limiting value of 0.3 mm specified in EC 2 (2004). As expected, the stiffness of foam concrete beams was much lower than NWC and LWAC beams and consequently, there was greater deflection of foam concrete beams at service load. More importantly, the ductility index of foam concrete beams was lower than NWC and LWAC beams, and sufficient ductility can only be achieved with very low tension reinforcement ratio, attributed to the more brittle behaviour of foam concrete. It should be noted that the foam concrete beams from Lim (2007) showed serious shrinkage cracks on the sample surface even prior to testing and these cracks would influence the performance. Prolonged water curing process could reduce shrinkage cracks and improve flexural performance of foam concrete beams, although it was still inferior to corresponding NWC and LWAC beams. Tan et al. (2005a) also investigated the flexural behaviour of 25 MPa foam concrete beams and similar conclusions to Jones and McCarthy (2005a) and Lim (2007) were obtained.

2.3 Carbon Nanofibers Enhanced Concrete

2.3.1 Introduction

Nowadays, the usage of carbon nanofibers (CNFs) and carbon nanotubes (CNTs) as nanoreinforcement in cementitious material has become increasingly popular to further enhance engineering properties of concrete (Sanchez and Sobolev, 2010). The first synthesis of CNFs was in 1889 and it was recorded as filamentous carbon at that time (Edison, 1892) while the first discovery and report of CNTs were made by Iijima

(1991) from Japan. Both CNFs and CNTs are highly structured graphene ring-based materials but their morphologies are quite different. The graphene sheets in CNFs are perpendicular to the fiber axis in the stacked form or aligned at an angle to the fiber axis in the herringbone form as shown in Figure 2.1 (a) and (b), respectively. The structure of CNTs is described by rolling of graphene sheet into a cylindrical tube. CNTs can be divided into single-wall CNTs (SWCNTs) and multiple-wall CNTs (MWCNTs) according to whether they consist of a single rolled graphene sheet or multiple concentrically rolled graphene sheets (see Figure 2.1 (c) and (d)).

Figure 2.1 Representation of CNFs (a) Stacked form; (b) Herringbone form and CNTs (c) SWCNTs; (d) MWCNTs

Both CNFs and CNTs present extraordinary mechanical, thermal and electrical properties due to the carbon to carbon sp^2 bonding (Peyvandi et al., 2014). Table 2.3 summarises the dimensions, mechanical, thermal and electrical properties:

Table 2.3 Dimension, mechanical, thermal and electrical properties of CNFs and CNTs

Properties	CNFs	CNTs
Diameter (nm)	50 ~ 200 (Metaxa et al., 2010)	SWCNTs: 0.4~10 MWCNTs: 4~100 (Fraga et al., 2004)
Length	From nanometer to centimeter (depending on manufacturing) (Teo et al., 2003)	
Aspect ratio (length/diameter)	> 100 (Poveda and Gupta, 2016)	> 1000 (Xie et al., 2005)
Density (kg/m ³)	1900 ~ 2100 (Endo et al., 2001)	1300 ~ 1400 (Collins and Avouris, 2000)
Tensile strength (GPa)	3 ~ 7 (Zhou et al., 2009)	60 ~ above 100 (Teo et al., 2003)
Young's modulus (TPa)	0.025 ~ 0.4 (Metaxa et al., 2010)	1 ~ 1.4 (Teo et al., 2003)
Elongation at failure (%)	NA	12 ~ 30 (Siddique and Mehta, 2014)
Thermal conductivity (W/mK)	1000 (Yu et al., 2006)	SWCNTs: 1750 ~ 5800 (Hone et al., 1999) MWCNTs: > 3000 (Kim et al., 2001)
Electrical resistance ($\Omega \cdot m$)	5×10^{-7} (Heremans, 1985)	1×10^{-6} (Yakobson and Avouris, 2001)

It is found in Table 2.3 that the mechanical, electrical and thermal properties of CNTs are clearly greater than those of CNFs. This is because the rolled graphene tubes of CNTs which are parallel to the fiber axis form the highly crystallized carbon structure (Teo et al., 2003). This structure has fewer exposed edges and dangling bonds on the surface of CNTs that lead to inert properties and weaker physical bonds with other materials (Teo et al., 2003). Conversely, the many exposed edges on the surface of CNFs provide desired sites to be functionalized and can form more effective physical or chemical bonds with other molecules (Peyvandi et al., 2014). Meanwhile, the large dimensions and relatively inferior properties of CNFs lead to lower production cost and high availability. The cost of vapour grown CNFs is approximately 100 times

lower than SWCNTs (Kang et al., 2006). Consequently, CNFs are more commercially viable for mass production compared to CNTs.

CNFs and CNTs have already been widely studied and applied with polymers in composite materials to improve their mechanical and electrical properties. Recently, increasing efforts were focused on their employment in cement-based materials. It was reported that CNFs/CNTs not only increase the mechanical properties and durability of cement-based materials, but also provide some novel properties such as electromagnetic field shielding and self-sensing (Sanchez and Sobolev, 2010). However, potential applications of CNFs/CNTs face several great challenges. The uniform and stable dispersion of CNFs/CNTs in concrete, and their effective interfacial bonding with cementitious materials are the two most critical issues. In addition, mass production and commercial production cost are important challenges hindering the applications of CNFs and CNTs. Only CNFs are introduced here because they are the only carbon nanomaterials used in CNF-LCC for present research.

2.3.2 Growth Mechanism and Morphology of CNFs

The morphology of carbon nanofibers is determined by the growth mechanism. However, different growth mechanisms depend on various fabrication processes. There are several methods to produce CNFs such as arc-discharge and laser-ablation methods but both cannot ensure quality. From the application point of view, catalytic

chemical vapor deposition (CVD) method is the most promising technique to produce CNFs that allows precise control over the length and diameter of CNFs (De Jong and Geus, 2000). In general, the CVD method consists of catalyst nanoparticles (usually Fe, Cu, Ni or alloy), carbon-containing gases (hydrocarbon gas) and heat. The process of CVD method is shown in Figure 2.2. Under high temperature, the carbon feedstock is adsorbed and decomposed on the catalyst/gas interface (1) of metal particles. The hydrogen and carbon atoms are released separately from the carbon feedstock gases. The molecular hydrogen will be removed from this interface, whereas the carbon atoms will dissolve in the metal particle and diffuse through the bulk of the catalyst (2). After diffusion, the carbon atoms will precipitate on the catalyst/solid carbon interface (3) in the form of fibrous structures. The orientation of this surface will determine the structure of CNFs.

Figure 2.2 Schematically showing the three distinct regions during the catalytic process (1) Catalyst/gas interface; (2) Bulk of catalyst; (3) Catalyst/solid carbon interface

The graphite sheets are precipitated parallel to the surface of faceted catalyst particle (Boellaard et al., 1985). Hence, the diameter of produced CNFs, as well as the angle between the graphite sheets and the fiber axis, is determined by the geometric facets of the catalyst particles. The length of generated CNFs is determined by the duration of catalytic reaction, that is, a longer duration will lead to longer fiber length (Fonseca et al., 1998). Based on the nature of catalyst particles, carbon feedstock and temperature, CNFs can be generally divided into *stacked* and *herringbone* structures according to the angle of graphene layers with respect to the fiber axis (Poveda and Gupta, 2016). In stacked form, the graphite sheets are stacked in a direction parallel to the base of the catalyst and perpendicular to the fiber axis (Figure 2.1 (a)). The stacked structure is usually straight and thick (50 to 300 nm) because catalysts with steady state and of larger size are used (Zheng et al., 2004). Stacked CNFs present a turbostratic structure and the spacing between the graphite sheets is approximately 0.34 nm from accurate measurements (Rodriguez et al., 1995). The graphite platelets of herringbone form CNFs are aligned at an angle to the fiber axis (Figure 2.1 (b)). Similar to the stacked structure, the spacing between the platelets is 0.34 nm (Rodriguez et al., 1995).

2.3.3 Dispersion of CNFs

The outstanding properties of CNFs can be used to improve engineering behaviour of cementitious composites. However, the successful transfer of their excellent

properties to the composites can only be realized when the CNFs are dispersed uniformly and have effective bond with the cement-based materials. Unfortunately, untreated CNFs have a strong tendency to form agglomeration due to the large surface area and Van der Waals force (Parveen et al., 2013). The hydrophobic property of CNFs makes their dispersion and handling in solvents or matrices difficult (Sanchez and Sobolev, 2010). The “dispersion” represents the procedure of deagglomeration of nanomaterials by mechanical means and subsequently stable distribution in solvents or composites by chemical methods (Parveen et al., 2013). Poor dispersion results in heterogeneous distribution and creates many defect sites which will reduce the efficiency of CNFs (Xie et al., 2005) and even degrade the performance of the composites. The poor dispersion of CNFs in water is shown by transmission electron microscope (TEM) images in Figure 2.3 (Cwirzen et al., 2008, Konsta-Gdoutos et al., 2010a). Even if uniform and stable dispersion is realised, CNFs still cannot show their advantages without effective interfacial bonding with surrounding composites. The instinctively stable structure of carbon atoms of CNFs makes them essentially chemically inert and their interfacial bonds with composites are relatively weaker.

Figure 2.3 TEM images of poor dispersion of carbon nanomaterials in water (Konsta-Gdoutos et al., 2010a, Cwirzen et al., 2008)

Directly dispersing CNFs within cementitious composites is infeasible because of the high viscosity and fast hardening of cement-based material (Gopalakrishnan et al., 2011). The general approach is to disperse CNFs in water and subsequently mix this nanofibers/water solvent with cement using a traditional mixer. The technological process of dispersing CNFs in water commonly contains mechanical and chemical approaches. Only the mechanical approaches can directly disperse CNFs in water and the chemical methods are mainly responsible for rendering solubility, maintaining dispersion stability, and creating effective interfacial bonds of CNFs. Both mechanical and chemical approaches are indispensable for a successful dispersion process.

2.3.3.1 Mechanical approaches

The main mechanical approach is through ultrasonic treatment which is generally conducted in an ultrasonic bath (Ma et al., 2010). The mechanism of this technique

is to propagate the ultrasound wave to the liquid medium and then convert it to mechanical vibrations which result in formation and violent collapse of microscopic bubbles. This process is known as cavitation that creates millions of shock waves and releases high level of energy (Parveen et al., 2013). The produced shock waves can gradually separate individual CNFs from the outer part of the agglomeration. The ultrasonication method is ideal for dispersing carbon nanofibers in solvents with low viscosity, such as water (Ma et al., 2010). Although ultrasonication can be used to directly disperse the CNF bundles, the CNF solvent will experience sedimentation over a period of time if there is no chemical treatment of CNFs to maintain stability.

2.3.3.2 Chemical approaches

Covalent Functionalisation

Covalent functionalisation is the most general means not only to improve the dispersion of carbon nanomaterials but also to enhance the interfacial bonds of CNFs with surrounding matrices. The functionalisation process is to oxidise the nanofiber surface by utilising the functionalising agents and then generate the functional groups on the surface of CNFs based on covalent bonding of carbon atoms. The functionalising agents are commonly aggressive oxidising chemical reactivity, such as strong acid, strong oxidants and plasma (Ma et al., 2010). The functional groups include amino (-NH₂), carboxylic acid (-COOH) and hydroxyl (-OH) groups (Zhang and Chen, 2004) which depend on the oxidising chemical used.

The solubility of CNFs is rendered by attaching functional groups which have the hydrophilic property (Ma et al., 2010). Simultaneously, the Van der Waals force can be reduced due to the presence of the functional group and individual CNF can be easily separated from the agglomeration by ultrasonication (Balasubramanian and Burghard, 2005). Moreover, the functionalised nanofibers can produce strong interfacial bonds with cementitious matrix due to covalent bonding between the functional groups and hydration products of cement such as calcium silicate hydrate and calcium hydroxide. Li et al. (2005) showed the formation of covalent bonding with the help of Fourier-transform infrared spectra and provided the mechanism of the chemical reaction between the carboxylic acid groups ($-\text{COOH}$) and the hydration products as shown in Figure 2.4.

Figure 2.4 Chemical reactions between carboxylated carbon nanomaterial and cement hydration products (C-S-H and Ca(OH)_2) (Li et al., 2005)

However, the functionalisation methods have some side-effects due to the case of aggressive oxidising chemicals. The violent oxidising reaction on the surface of carbon nanomaterial unavoidably creates a mass of defects. Combined with ultrasonication, the nanofibers can be easily broken into fragments which are called “carboxylated carbonaceous fragments” resulting in poor mechanical and electrical properties (Nasibulina et al., 2012). Meanwhile, the aggressive oxidants themselves are environmentally unfriendly.

Surfactant

Unlike covalent functionalization method which deteriorates CNFs, surfactant method is more attractive because they are physically absorbed on the CNF surface without altering the inherent structures and properties (Nasibulina et al., 2012).

The amphiphilic and charged surfactants are suitable for CNF/water soluble solution. An individual surfactant molecule consists of a hydrophilic group (head group) and a hydrophobic group (tail group). The hydrophobic tail groups of surfactants will attach to the surface of CNFs based on a strong hydrophobic attraction force (Vaisman et al., 2006). This hydrophobic attraction force is a non-covalent attraction force (Bystrzejewski et al., 2010) which can maintain the properties of carbon nanomaterials (Rastogi et al., 2008). Surfactants can reduce the surface tension of water and render carbon nanofibers soluble in water due to the presence of hydrophilic groups (Parveen et al., 2013). In addition, the same charges on the head

group of the surfactants can produce steric and/or electrostatic repulsions between the agglomerated CNFs which result in a stable dispersion (Vaisman et al., 2006). However, the cost of surfactants is very high, which makes mass production uneconomical.

2.3.4 The effect of CNFs on concrete

2.3.4.1 Acceleration of hydration process

The early research from Makar et al. (Vera-Agullo et al., 2009) indicated that the hydration of cement can be accelerated by adding properly dispersed CNFs. The higher CNFs content can increase the hydration degree at 7 and 28 days although to a limited extent. The same conclusion was obtained by SEM and Vickers hardness measurements (Konsta-Gdoutos et al., 2010a). CNFs can work as nucleation sites due to their high surface energy and the hydration products will deposit on their surface (Sanchez and Sobolev, 2010). During this process, a greater amount of calcium silicate hydrate (C-S-H) will be produced in the early stage and consequently, the mechanical properties are improved.

2.3.4.2 Filler function

Another function of CNFs in cementitious composites is working as fillers to reduce the porosity of microstructure, especially the interfacial transition zone between cement paste and aggregates. Li et al. (2005) studied the effect of CNFs on the

porosity of cement-based material with the help of the mercury intrusion porosimetry. The test results showed that total porosity was decreased by 64% compared to the control mortar. Meanwhile, the pore size distribution also shifted to lower level because the volume of both macropores (pore size > 50 nm) and mesopores (pore size < 50 nm) were decreased and the mean pore diameter was reduced from 30 nm to 17.4 nm. The filler function is more significant in ultra-high performance concrete (UHPC) which is a concrete material with ultra-high packing density. CNFs can further improve the packing density of UHPC by filling into these nanoscale and microscale regions as shown in Figure 2.5.

Figure 2.5 Filler function of CNFs in UHPC

The reduced porosity will improve not only the mechanical properties but also the durability of cementitious composites. Han et al. (2013) measured the transport properties such as sorption and permeability of cement paste enhanced by CNFs. These values were reduced significantly compared to the plain cement paste. In

addition, Konsta-Gdoutos et al. (2010b) reported that the autogenous shrinkage of cement paste was decreased by about 30% due to the addition of carbon nanomaterials reducing the volume of the capillary pores.

2.3.4.3 Delay of the nano cracks growth

Cement-based materials are always characterized by low tensile strength, toughness, and ductility which make them vulnerable to cracking. The cracking development in concrete starts from isolated nano cracks interconnecting to form microcracks, and finally propagating into macrocracks. Both the macrofibers and microfibers such as polymeric and steel fibers are effective to increase the post-cracking behaviour because of their larger size and spacing in concrete. However, CNFs are capable of preventing crack growth from nano cracks to microcracks due to their large aspect ratio, nano-sized dimension and outstanding mechanical properties (Parveen et al., 2013). CNFs can act as bridges between nanoscale cracks or voids (see Figure 2.6) to improve cracking resistance and further increase tensile strength. Meanwhile, the damaged cementitious composite samples enhanced by CNFs also show better post-testing mechanical integrity which reflects increased ductility as shown in Figure 2.7 (Sanchez et al., 2009).

Figure 2.6 SEM images of a nano crack bridged by CNFs (Hou and Reneker, 2004)

Figure 2.7 Damaged test samples without (a) and with (b) CNFs (Sanchez et al., 2009)

2.3.5 Carbon Nanofibers Enhanced Foam Concrete

Foam concrete and CNFs enhanced concrete have been introduced in Section 2.2 and 2.3. However, the hybrid use of carbon nanofibers with foam concrete deserves further elaboration. CNFs enhanced foam concrete can keep the advantages of foam concrete and the low mechanical properties and durability may be improved by CNFs due to their modification effect on foam concrete microstructure.

Yakovlev et al. (2006) studied the effect of carbon nanotubes on foam concrete. From the addition of 0.05% of initial mass of CNTs, the compressive strength of foam concrete was increased 70% from 0.18 MPa to 0.31 MPa and thermal conductivity was reduced 20% from 0.07 W/mK to 0.056 W/mK. The enhanced mechanical properties and reduced thermal conductivity were attributed to modifications of the microstructure. In traditional foam concrete, the intensive percolation on the pore walls increases the possibility of combining small foam bubbles into larger bubbles, which result in a wider range of pore size distribution. Fortunately, the percolated walls can be repaired by introducing CNTs which play the role of centres for directive crystallisation resulting in the development of a fibrillary structure on the pore walls. Then the much more homogeneous pore wall structure ensures smaller and more uniform pore size distribution in foam concrete. Consequently, the mechanical properties were enhanced and thermal conductivity was reduced. This pioneering study indicates the potential reinforcing effect of carbon nanomaterials on foam concrete.

2.4 Summary

Conventional foam concrete is mainly used for non-structural applications due to its low mechanical properties. The first challenge is to obtain the acceptable compressive strength for structural usage at a relatively low density, i.e., high strength to density ratio. The following problems are the extremely low tensile strength and elastic

modulus, which influence the performance of foam concrete structures within the serviceability limit. In addition, durability (permeation properties) and shrinkage resistance of foam concrete are lower than NWC and they can be improved with reduced density due to increased foam volume and decreased amount of cement paste. However, lowering cement paste content in foam concrete will render the mechanical properties undesirable for structural requirements. Therefore, this is another challenge for foam concrete to achieve reliable mechanical properties, durability and shrinkage resistance at the same time. Prior to considering structural applications, bond behaviour between foam concrete and steel reinforcement is critical but the related studies are scarce and foam concrete seems to have poor bond strength due to its low mechanical properties. Moreover, studies of thermal properties of foam concrete under high temperature are insufficient but they are important for evaluating fire resistance of a structural material. Finally, the limited structural experiments show that structural performance of reinforced foam concrete is inferior to NWC and LWAC.

Carbon nanofibers have been shown to increase the engineering properties of concrete due to their outstanding characteristics. However, as nanomaterials, the cost of carbon nanofibers is high because of the complex production process and subsequent dispersion process, especially by the chemical approaches. Therefore, the study of CNFs enhanced concrete mainly focused on the material level where only a

small volume of CNFs is required. It is necessary to extend the study of CNFs in concrete to a broader field including structural applications when high-quality and economical mass production of CNFs can be achieved.

In this study, CNF-LCC is a new type of foam concrete for structural applications. It was developed by the methods of UHPC and nano-engineering based on CNFs. The CNFs used in this study were provided by ceEntek Pte Ltd with excellent dispersion and reduced cost as a result of improved production processes of CNFs. CNF-LCC solved the challenges and problems in traditional foam concrete for structural uses. CNF-LCC is comparable or even performs better than NWC and LWAC. The following chapters show the performance along with a comprehensive analysis of CNF-LCC in terms of mix design, short- and long-term properties, bond behaviour with steel reinforcement and flexural performance of reinforced CNF-LCC beams. Moreover, the effect of CNFs on corresponding performance is investigated and the outcomes will be discussed based on the modified micro- and nano-structure.

CHAPTER 3 MIX DESIGN AND MECHANICAL PROPERTIES

3.1 Introduction

Material properties are the basic information to evaluate the performance of a novel type of building materials and are essential parameters for structural analysis and design. In this chapter, the mix design of CNF-LCC in the present study is introduced.

The optimisation concept, mix components and corresponding material amount, and preparation process of CNF-LCC were presented and the fresh properties were determined according to the standard code. Afterwards, the mechanical properties of hardened CNF-LCC that influence the short-term structural performance of reinforced CNF-LCC members were measured and reported. The measured mechanical properties included compressive strength, flexural tensile strength and toughness, elastic modulus and compressive stress-strain relationship. In addition, the mechanical properties of CNF-LCC were compared with those of NWC & LWAC to evaluate its potential for structural applications. It should be mentioned that all the data of NWC & LWAC used for comparison were from standard codes because they represent common values while actual results may vary depending on the mix design. Moreover, the effect of CNFs on the microstructure and mechanical properties was also discussed by comparing CNF-LCC with the control samples LCC (matrix without CNFs).

3.2 Mix design

3.2.1 ceUHPC base mix

The optimisation concept of CNFs enhanced UHPC (ceUHPC) was based on the studies by Chen et al. (2016) to achieve improved mechanical properties and optimum workability of the fresh mixture. A low water-to-cement ratio and high packing density of solid material were designed to ensure high mechanical properties of resulting UHPC. The workability of fresh ceUHPC was critical to the stability of introduced foam when producing CNF-LCC and it was optimised by the optimal particle grading of solid materials and the usage of superplasticizer. The optimum workability could also minimise the entrapped air to increase the mechanical properties of hardening ceUHPC. Furthermore, a low dosage of highly dispersed CNFs through ultrasonication without chemical approaches was introduced to modify and strengthen the microstructure of ceUHPC. The mix designs of ceUHPC and control UHPC base mix (without CNFs) are presented in Table 3.1.

Table 3.1 Mix Design of ceUHPC and UHPC (unit: a relative portion in weight)

Specimen	Cement	Graded quartz Sands	Water	SP	CNFs
ceUHPC	1	0.36	0.24	0.006	0.0007
UHPC	1	0.36	0.24	0.006	0

3.2.1.1 Cement

The phase composition of used CEM 1 Ordinary Portland cement in this study was measured by X-ray diffraction and the results are listed in Table 3.2.

Table 3.2 Phase composition in Ordinary Portland cement

Chemical components	C ₃ S	C ₂ S	C ₃ A	C ₄ AF	Equivalent alkalis
Percentage by weight	70%	12%	5%	10%	0.82%

3.2.1.2 Graded sands

Graded quartz sands with a particle size of 100 to 600 µm particle size range were used as very fine aggregates. Particle grading of cement and quartz sands was performed using Horiba Laser Particle Size Analyzer. An optimal packing density was achieved by considering the modified Andreasen and Andersen (A&A) model as follows:

$$P(D) = \frac{D^q - D_{min}^q}{D_{max}^q - D_{min}^q} \quad (3.1)$$

where D is the particle size (µm), $P(D)$ is the fraction of total solids smaller than size D , D_{max} is the maximum particle size (µm), D_{min} is the minimum particle size (µm), and q is the distribution modulus. Figure 3.1 shows the particle size distribution of optimal grading of cement particles and quartz sands used which is close to the ideal modified A&A model curve. The particle grading of solid material will not only influence the flowability of the base mix but also the stability of introduced foam when

blended with the base mix (Ramamurthy et al., 2009). To illustrate this, a poor grading of solid material (the grading curve relatively deviates from the ideal modified A&A model curve in Figure 3.1) was also prepared for mixing and the resulting flowability of base mix and the stability of CNF-LCC were compared with those produced by optimal grading solid material in Section 3.2.2. The parameters in Equation 3.1 for the idea, optimal and poor grading curves are listed in Table 3.3

Table 3.3 Values of D_{max} , D_{min} and q for ideal, optimal and poor grading curves

Grading curves	D_{max} (μm)	D_{min} (μm)	q
Ideal	300	38	
Optimal	250	20	0.23 (Yu et al., 2014)
Poor	280	10

Figure 3.1 Particle size distribution of optimal and poor grading of solid material

3.2.1.3 CNFs

A new type of CNFs was produced by ceEntek based on catalytic chemical vapor deposition (CVD) method and was synthesized with a wide range of diameters and lengths. The properties of resulting CNFs and CNF suspension are listed in Table 3.4. Scanning electron microscope (SEM) images of CNFs are shown in Figure 3.2 (a). The herringbone form (graphite sheets are aligned at an angle to the fiber axis) of CNFs was observed by transmission electron microscope (TEM) as shown in Figure 3.2 (b). The dispersion of nanomaterial is a challenge due to its nano-size in nature and it tends to agglomerate and shows sedimentation after a short period of time. The dispersion process through ultrasonication was earlier documented by Chen et al. (2016). There is no sedimentation of CNFs found after 3 days and the CNFs colloid still appear in stable homogenous form due to the high surface zeta potential of CNFs as summarised in Table 3.4. This makes it possible for CNFs to be dispersed in water through a physical process without addition of chemical modification and/or surfactant (Chen et al., 2016).

Table 3.4 Properties of CNFs and CNF suspension

CNF properties					CNF suspension	
Purity (%)	Diameter (nm)	Zeta Potential (mV)	Length (μm)	Surface area (m^2/g)	Concentration	pH
>95	10 ~ 100	+33	10 ~ 20	120 ~ 130	0.3wt%	5.6 ~ 6.2

(a)

(b)

Figure 3.2 (a) SEM pictures of CNFs; (b) TEM pictures of herringbone form CNFs

3.2.1.4 Superplasticizer

A polycarboxylate-based superplasticizer with 35% solid content by weight was used as a water-reducing admixture without affecting the rheological properties when CNFs were incorporated.

3.2.2 Preparation of CNF-LCC/LCC

Preparation of CNF-LCC/LCC is according to the pre-foaming method as mentioned in Section 2.2.1.2. The ceUHPC/UHPC base mix and micro-foam were produced simultaneously and subsequently, the obtained micro-foam was introduced into the base mix to achieve a density of $1500 \pm 50 \text{ kg/m}^3$. For the production of homogeneous foam, 60mmol/L of betaine (NS B) foaming agent which complied with DIN EN 206-1 (2001) was used and added to the dispersions, homogenized for 90 seconds and foamed simultaneously. The foaming was induced with a continuous dynamic stirrer (Hobart). The production of micro-foam was performed in accordance with the recommendations given by Krämer et al. (2015). Before blending the base mix with the micro-foam, flow table tests were performed in accordance with ASTM C230 (2014a) to evaluate the flowability of the base mix. Based on the discussion in Section 3.2.1, two kinds of base mix produced by optimal and poor grading of solid material were conducted to investigate the effect of solid material grading on the viscosity of the base mix. It was found that optimal grading achieved a higher flow expansion of $320 \pm 10 \text{ mm}$ compared to poor grading with a $300 \pm 10 \text{ mm}$ flow expansion. The

flow expansion indicated the viscosity of base mix to allow micro-foam to be injected into the matrix. The target flow expansion of ceUHPC base mix is at 320 mm (Figure 3.3 (a)).

(a)

(b)

(c)

Figure 3.3 Typical flow spread of (a) ceUHPC base mix (320 mm), (b) LCC (270 mm), and (c) CNF-LCC (270 mm)

After mixing with micro-foam, the CNF-LCC mixture with optimal and poor grading respectively was cast to 50 mm cube specimens and the hardened cube cross-sections were captured by an optical microscope and analysed by an image processing software to characterise the micro-foam structure in the matrix as shown in Figure 3.4. Optimal grading with a much higher flow expansion showed a more homogeneous micro-foam structure with diameter ranging from 0.1 mm to 0.2 mm in the matrix (Figure 3.4 (a)) while poor grading with a lower flow expansion showed more merged micro-foam with diameter ranging from 0.5 mm to 1.0 mm in the matrix (Figure 3.4 (b)). Higher viscosity mixture caused greater damage to the micro-foam in the matrix. Hence, all the CNF-LCC and LCC specimens discussed in later chapters were produced by optimal grading of the solid material. The typical flow

expansion of both LCC and CNF-LCC is at 270 mm as shown in Figure 3.3 (b) and (c). All specimens were demoulded after 1 day of casting and air-cured at approximately 23 °C in a humidity-controlled chamber room.

(a)

(b)

Figure 3.4 Optical microscope of hardened cube cross-sections (a) optimal grading + micro-foam; (b) poor grading + micro-foam

3.3 Mechanical properties

3.3.1 Compressive strength

The compressive strength at 1, 7 and 28 days was obtained on standard cylinders ($\Phi 150 \times 300$ mm) and the procedure complied with BS EN 12390-3 (BSI, 2011). At the same time, 150 mm cube samples from the same batch were cast to determine the 28-day cube strength.

The cylinder compressive strength development with curing time of CNF-LCC and LCC is presented in Figure 3.5. Ramamurthy et al. (2009) summarised the 28-day compressive strength of foam concrete with the density between 240 and 1900 kg/m³ from the literature and all the specimens showed much lower compressive strength than both LCC and CNF-LCC produced by the author. This is attributed to the UHPC base mix creating high strength and denser borders (pore walls) in LCC and CNF-LCC. Besides, LCC and CNF-LCC also show significantly higher compressive strength than industrial cellular concrete (Schauerte and Trettn, 2012) as shown in Figure 3.6. It can be observed from Figure 3.5 that CNFs increase the compressive strength at 1, 7 and 28 days by 18.5%, 16.5%, and 12.7%, respectively. This is attributed to the filler function of CNFs to make the structure of pore walls denser by reducing the size and volume of capillary pores and microvoids (Li et al., 2005, Peyvandi et al., 2014), especially in the interfacial transition zone (ITZ) (Sanchez and Sobolev, 2010). Moreover, a stable dispersion of CNFs is critical to the improvement

of strength (Chen et al., 2016) due to reduced CNF-free volume of the base mix. The results indicated that 1-day compressive strength had the highest increase, which agreed with the conclusion from Sanchez and Sobolev (2010). They reported that CNFs could work as nucleation sites due to their high surface energy and the hydration products would deposit on their surface, during which a higher amount of calcium silicate hydrate (C-S-H) could be produced in the early hydration stage.

Conventionally, the cylinder compressive strength is lower than the cube strength due to the unavoidable confinement effect of testing of the latter. Therefore, there is a factor to convert the cylinder compressive strength to equivalent cube strength. For NWC, this factor ranges from 1.3 for low strength to 1.04 for high strength concrete (Aroni, 1993) and an average value of 1.25 has been applied for many years. In the case of LWAC, the value of this factor is reduced to 1.11 for conservatism (BSI, 2000).

An attempt was made to look for the conversion factor of CNF-LCC from cylinder to cube compressive strength. The 28-day compressive strength of CNF-LCC tested from 150 mm cubes was 32.1 MPa with 0.81 standard deviations. Therefore, the conversion factor of CNF-LCC was equal to $32.1/30.2 = 1.06$ which was lower than NWC & LWAC and quite close to 1.0. Gyengo (1938) studied the influence of several parameters on the conversion factor of cylinder to cube compressive strength. One of the parameters was the aggregate coarseness and their experimental results showed that the factor was closer to 1.0 with decreasing size of aggregates. This may

explain the abovementioned compressive strength test results because only fine quartz sands were used in the system of CNF-LCC resulting in very low coarseness of aggregates.

Figure 3.5 Cylinder compressive strength development of CNF-LCC and LCC

Figure 3.6 Comparison of compressive strength of industrial cellular concrete (Schauerte and Trettin, 2012), LCC and CNF-LCC

3.3.2 Flexural tensile strength and toughness

Flexural tests at 28 days were conducted on 40×40×160 mm prisms in accordance with ASTM C348 (2014b). Load-displacement graph was obtained from the test and the load was converted to stress at the extreme tension fibers of the prism according to Euler-Bernoulli elastic beam theory. Flexural tensile strength is equal to the peak stress in the graph and flexural toughness is generally defined as energy absorption capacity which is equal to the area under the load-deflection curve for bending (Tyson et al., 2011). The 28-day flexural tensile strengths of CNF-LCC and LCC are compared in Table 3.5. In addition, the flexural strengths of NWC and LWAC with similar compressive strength from major codes (Table 3.6) are also given in Table 3.5.

Table 3.5 28-day flexural tensile strength of CNF-LCC, LCC, NWC, and LWAC

Concrete	CNF-LCC	LCC	NWC ($f_{cm}=30$ MPa)			LWAC ($f_{cm}=30$ MPa)		
Flexural tensile strength (MPa)	4.8 (0.25*)	3.5 (0.29*)	ACI (2011)	EC 2 (2004)	CEB-FIP (2010)	ACI (2011)	EC 2 (2004)	CEB-FIP (2010)
			3.4	3.5	26.9	2.7	2.9	12.8

Noting: * represents the standard deviation

Table 3.6 Expressions of flexural strength of NWC and LWAC from major codes

Codes	ACI (2011)	EC 2 (2004)	CEB-FIP (2010)
NWC f_r	$0.62\sqrt{f'_c}$	$\max\{(1.6 - h/1000)f_t; f_t\}$ where $f_t = 0.3 \cdot f_{ck}^{2/3}$ for $f_{ck} \leq 50\text{MPa}$	$\frac{1 + 0.06 \cdot h^{0.7}}{0.06 \cdot h^{0.7}} \cdot f_t$ where $f_t = 0.3 \cdot f_{ck}^{2/3}$ for $f_{ck} \leq 50\text{MPa}$
LWAC f_{lr}	$0.47\sqrt{f'_c}$	$\eta_1 f_r$ where $\eta_1 = 0.4 + \frac{0.6\rho_d}{2200}$	$\eta_1 f_r$ where $\eta_1 = 0.4 + \frac{0.6\rho_d}{2200}$

Noting: f_r = flexural tensile strength of NWC in MPa; f_{lr} = flexural tensile strength of LWAC in MPa; f'_c = cylinder compressive strength in MPa; f_{ck} = characteristic cylinder compressive strength in MPa; f_t = direct tensile strength of NWC in MPa; f_{lt} = direct tensile strength of LWAC in MPa; h = total member depth in mm; ρ = unit weight of concrete in kg/m³.

It was reported that the tensile strength of foam concrete is lower than NWC and LWAC of the same compressive strength (Ramamurthy et al., 2009, Jones and McCarthy, 2005a). However, both CNF-LCC and LCC showed higher or equal flexural strength compared with NWC and LWAC; in particular, specimens for CNF-LCC, the flexural strength was 39.1% and 71.4% higher than NWC and LWAC, respectively. This may be attributed to the high packing density of the base mix which increased the shear capacity between sand particles and cement paste (Jones and McCarthy, 2005a). By adding CNFs, the flexural strength of LCC was increased by

37.1%. Toughness which represented by the area under the stress-displacement curve (Figure 3.7) showed that CNF-LCC had 50.8% higher absorption capacity compared to LCC. The homogenous and stable dispersion of CNFs created more fiber-matrix interfaces where CNFs can work as bridges to arrest propagation of nano- and micro-cracks, resulting in improved flexural strength and toughness (Li et al., 2005, Chen et al., 2016). Meanwhile, the bridging capacity of CNFs can be enhanced by the functionalized surface of CNFs, which can produce strong interfacial bonds with cementitious matrix due to covalent bonds between functional groups and hydration products (Li et al., 2005).

Figure 3.7 Flexural stress and displacement curve of CNF-LCC and LCC

3.3.3 Elastic modulus

In this study, the modulus of elasticity E is taken as secant value between zero and 0.4 times compressive strength in the stress-strain curve which is often used for structural design in EC2. The 28-day elastic modulus tests strictly complied with BS EN 12390-13 (2013). The test specimens were $\Phi 150\text{mm} \times 300\text{mm}$ cylinders according to BS EN 12390-1 (2012b). There were 3 specimens tested and the average results are listed in Table 3.7. The expression for modulus of elasticity of NWC and LWAC from major codes are summarised in Noting: * represents the standard deviation

Table 3.8. These predicted values of NWC and LWAC ($f_{cm} = 30 \text{ MPa}$) are compared with the measured elastic modulus of CNF-LCC & LCC in Table 3.7.

Table 3.7 28-day elastic modulus of CNF-LCC, LCC, NWC, and LWAC

Concrete	CNF-LCC	LCC	NWC ($f_{cm}=30 \text{ MPa}$)			LWAC ($f_{cm}=30 \text{ MPa}$)		
Elastic modulus (GPa)	14.5 (0.45*)	12.6 (0.51*)	ACI (2011)	EC 2 (2004)	CEB-FIP (2010)	ACI (2011)	EC 2 (2004)	CEB-FIP (2010)
			27.7	30.6	26.9	14.0	14.6	12.8

Noting: * represents the standard deviation

Table 3.8 Expressions of elastic modulus of NWC and LWAC from major codes

Source	ACI (2011)	EC 2 (2004)	CEB-FIP (2010)
NWC E_c	$0.043\rho_d^{1.5}f_c^{0.5}$ (MPa)	$22(f'_c/10)^{0.3}$ (GPa)	$21.5 \cdot (0.8 + 0.2 \cdot \frac{f_{cm}}{88}) \cdot (\frac{f_{cm}}{10})^{\frac{1}{3}}$ (GPa)
LWAC E_{lc}	$0.043\rho_d^{1.5}f_c^{0.5}$ (MPa)	$E_c \cdot (\rho_d/2200)^2$ (GPa)	$E_c \cdot (\rho_d/2200)^2$ (GPa)

Noting: E_c = elastic modulus of NWC; E_{lc} = elastic modulus of LWAC; f'_c = cylinder compressive strength in MPa, ρ_d = unit weight of concrete in kg/m³.

As shown in Table 3.7, introducing CNFs improved the elastic modulus of LCC from 12.6 GPa to 14.5 GPa by 15.1%. However, the elastic modulus of CNF-LCC was only about half of NWC because the coarse granite aggregates used in NWC have higher elastic modulus and account for a large fraction, while CNF-LCC does not have coarse aggregates and the introduced air voids reduce elastic modulus. On the other hand, compared with traditional foam concrete whose elastic modulus is significantly lower than NWC and LWAC with values typically varying from 1.0 to 8.0 GPa for density between 500 and 1500 kg/m³ (Jones and McCarthy, 2005a, Ramamurthy et al., 2009), CNF-LCC increases the maximum value (8.0 GPa) by 81.3% and on par with LWAC. This is attributed to the ceUHPC used as the base mix in CNF-LCC. In cement paste, the elastic modulus of solid phase in ascending order

is porous phase, low stiffness C-S-H, high stiffness C-S-H, calcium hydroxide, followed by unhydrated phase (Mondal et al., 2008, Constantinides and Ulm, 2007). Compared with traditional foam concrete, the ceUHPC base mix in CNF-LCC was developed by low water/cement ratio and high packing density. Therefore, the porous phase should be significantly reduced and the unhydrated phase will exist. The nano-size of CNFs will effectively reduce capillary porosity and the porous phase will be further reduced. Moreover, by employing nanoindentation technology, it was observed that incorporation of CNTs or CNFs can increase the amount of high stiffness C-S-H (Konsta-Gdoutos et al., 2010b, Barbhuiya and Chow, 2017). From the point of structural applications, based on higher elastic modulus, the deflection of reinforced CNF-LCC beams under load should be lower than that observed in (Jones and McCarthy, 2005a, Tan et al., 2005a) to satisfy serviceability limit state.

3.3.4 Stress-strain relationship

In reinforced concrete structures, compressive stresses are essentially carried by concrete and tensile stresses by steel reinforcement. In addition to the compressive strength of concrete which is important to the design of structures, strain is also vital because it is associated with stress. Therefore, the complete stress-strain relationship of concrete under compression must be obtained to evaluate the response of loaded structural members.

Obtaining a complete stress-strain curve is challenging because the shape is significantly influenced by the testing conditions including the stiffness of testing machine, shapes and sizes of the specimens, specimen versus machine stiffness, strain rate, type of strain gauge, gauge length, and type of loading (Carreira and Chu, 1985).

The ascending branch up to the maximum compressive strength can be precisely measured by strain gauges at the center of the specimen. However, the challenge is to obtain a complete and accurate descending branch because the strain energy stored in the testing machine is released into the specimen once the unloading period starts.

This suddenly delivered energy may generate premature damage in the specimen which may result in a steep drop in the peak value of the stress-strain curve. In the case of LWAC, the reduction is even more prominent due to the lower stiffness and higher brittleness of LWAC. Several methods are proposed to reduce or eliminate the influence of strain energy release in the descending branch of the stress-strain curve.

One approach is to set up the specimen within a steel cylinder so that the applied load is always increased and the loading machine has no time to release the stored strain energy (Wang et al., 1978a). Another approach is to use a closed-loop servo-controlled hydraulic testing machine with a feedback control system to release the stored strain energy slowly and gradually. Otherwise, the simplest method is to choose a slow strain rate during the test.

Testing was conducted in accordance with BS EN 12390-13 (2013). The test specimens were $\Phi 150\text{mm} \times 300\text{mm}$ cylinders according to BS EN 12390-1 (2012b). Two concrete strain gauges with a gauge length of 60 mm were mounted on opposite faces and on the mid-depth of the specimen to monitor the strain variation in the ascending branch. A compressometer consisting of three linear variable differential transducers (LVDTs) were fixed to the middle one-third of the cylinder specimen to obtain the strain gauge readings in ascending branch and to trace the post-peak behaviour because crushing concrete will damage the strain gauges. The test set-up and machine are shown in Figure 3.8. Due to unavailability of the closed-loop servo-controlled hydraulic testing machine with a feedback control system, a slow and constant strain rate of 0.01mm/min was selected during the entire loading test to reduce the effect of strain energy released from the testing system during the unloading period. The cylinder specimen should be adjusted to ensure concentric axial load acting on the specimen. There were three specimens tested in each group and the average results are plotted in Figure 3.9.

Figure 3.8 Testing set-up and machine for compressive stress-strain curves

Figure 3.9 Compressive stress-strain relationship of CNF-LCC and LCC at 28 days

During the tests, it was observed that the strain gauges detached from the specimen after the peak value due to cracks forming on the surface. Meanwhile, the measured strain in the ascending branch by LVDTs and strain gauges coincided with each other

which implied the correct testing procedure. Both CNF-LCC and LCC showed brittle behaviour because a rapid failure occurred after the peak value and the descending branch of stress-strain curves could not be traced by LVDTs. LWAC also showed brittle performance exhibited by a steeper descending branch of the stress-strain relationship (Almusallam and Alsayed, 1995). In Eurocode 2 (2004), the descending branch of LWAC for structural design is not even present. This can be explained by the difference in energy absorption between NWC and LWAC proposed by Zhang and Gjrv (1990). In NWC, the aggregates are stronger than the cement paste and the latter is stronger than interfacial transition zone. Therefore, propagation of cracks is along the interfacial transition zone and the aggregates can efficiently restrain the cracks from developing. Both the aggregates and ITZ can meander the cracking progress, during which a considerable amount of energy is absorbed. However, for LWAC, the hygrol equilibrium between porous lightweight aggregates and porous cement matrix strengthens the interfacial transition zone and the lightweight aggregates become the weakest constituent due to their porous structure. Thus, the cracks propagate typically through the lightweight aggregates leading to brittle failure of LWAC. In the case of CNF-LCC & LCC, the porous structure of the matrix and the absence of coarse aggregates in the system result in lower energy absorption to delay the development of cracks. Meanwhile, the effect of strain energy released from the testing machine was still present although the strain rate was reduced to 0.01 mm/min. They are the main factors resulting in the brittle performance of CNF-LCC

& LCC. As shown in Figure 3.9, the stiffness of the stress-strain curve of LCC was improved by introducing CNFs and the maximum strain of CNF-LCC was 20.8% higher than LCC. Furthermore, compressive energy absorption (toughness) which was represented by the area under the stress-strain curve showed that CNF-LCC had 64.8% higher absorption capacity compared to LCC. These improvements were attributed to the decreased amount of low-stiffness phase and increased the amount of high-stiffness phase in cement paste by incorporating CNFs as explained in Section 3.3.3. Furthermore, CNFs can work as bridges to prevent the crack growth from nano cracks to microcracks, resulting in improved energy absorption capacity (Li et al., 2005, Chen et al., 2016, Parveen et al., 2013). Sanchez et al. (2009) also reported that the ductility of cementitious composites was increased due to the enhancement of CNFs, which was reflected by the better post-testing mechanical integrity.

It is necessary to obtain mathematical formulae describing the experimental stress-strain curves of CNF-LCC and LCC for structural analysis and design in later sections. The non-linear equation for LWAC provided from EC 2 (2004) was used to predict the stress-strain curve of CNF-LCC and LCC as follows:

$$\sigma_c = \frac{f_{cm} \cdot \left[k \cdot \left(\frac{\varepsilon_c}{\varepsilon_{cm}} \right) - \left(\frac{\varepsilon_c}{\varepsilon_{cm}} \right)^2 \right]}{1 + (k-2) \cdot \left(\frac{\varepsilon_c}{\varepsilon_{cm}} \right)} \quad (3.2)$$

$$k = 1.05 E_c \cdot \varepsilon_{cm} / f_{cm} \quad (3.3)$$

where σ_c is the concrete stress, ε_c is the concrete strain, f_{cm} is the compressive strength, ε_{cm} is the concrete strain at compressive strength, E_c is the secant elastic modulus. Figure 3.9 shows that Equation 3.2 agreed well with the experimental results. It should be mentioned that this equation in EC 2 (2004) does not have the descending branch for LWAC due to its brittle performance.

3.4 Summary

The concept of mix design optimisation and ingredients used for CNF-LCC were introduced in this chapter. This optimal mix design is used throughout the thesis to study the performance of CNF-LCC in later chapters. Afterwards, the mechanical properties of CNF-LCC were measured by standard test methods and the results showed that CNF-LCC was reliable for structural applications. Furthermore, CNFs produced different levels of improvement on these mechanical properties. The measured mechanical properties are the essential parameters for structural analysis and design in subsequent chapters. The conclusions of CHAPTER 3 are as follows:

1. CNF-LCC was ideally tailored from the ceUHPC based mix design by adding synthesis forming agent into the matrix to create a lightweight cementitious composite with $1500 \pm 50 \text{ kg/m}^3$ density.
2. Optimum particle grading obtained from the modified Andreasen and Andersen model achieved a target flow expansion. The flow expansion correlated with the

viscosity of the base mix; a higher flow expansion was desirable for micro-foam to be homogeneously blended into the matrix.

3. The 28-day compressive strength of CNF-LCC exceeded the conventional cellular concrete attributed to the high strength and dense pore walls by the usage of ceUHPC mix. In addition, CNFs improved the compressive strength at different curing ages due to the filler function and acceleration of hydration products. Moreover, a value of 1.06 was measured to represent the conversion factor of the cylinder to cube compressive strength of CNF-LCC

4. The 28-day flexural tensile strength of CNF-LCC was not only higher than traditional foam concrete but also higher than NWC and LWAC with similar compressive strength. Incorporating CNFs delayed the growth of nano- and micro-cracks resulting in dramatical improvement in flexural strength and toughness.

5. Although elastic modulus of CNF-LCC was lower than NWC, it was higher than traditional foam concrete and on par with LWAC because of the ceUHPC base mix. The existence of CNFs reduced the amount of porous phase and increase the amount of high-stiffness phase, which led to the improvement of elastic modulus.

6. CNF-LCC showed a similar brittle compressive stress-strain relationship to LWAC. However, CNFs can modify the compressive stress-strain performance in terms of

higher stiffness, larger maximum strain and significantly increased compressive energy absorption capacity.

Mechanical properties are the basic short-term properties for evaluating CNF-LCC for structural applications. Besides, thermal properties at room and high temperature are also very important for a structural material to evaluate its thermal insulation properties and fire resistance. In the next chapter, the thermal properties of CNF-LCC including thermal diffusivity, specific heat, thermal conductivity, thermal strain and phase transformations during heating will be introduced.

CHAPTER 4 THERMAL PROPERTIES OF CNF-LCC UNDER HIGH TEMPERATURE

4.1 Introduction

Foam concrete is always associated with excellent thermal insulation property and fire resistance as introduced in Section 2.2.5. Meanwhile, as a new type of structural material, it is necessary to study thermal properties of CNF-LCC so that the fire performance of the material can be determined quantitatively. In this chapter, thermal properties including thermal diffusivity, specific heat, and thermal expansion of ceUHPC, CNF-LCC and LCC are determined experimentally from room temperature to 800 °C. One-dimensional heat transfer tests using the ISO834 standard fire curve were carried out with CNF-LCC and LCC samples. The measured thermal properties were validated using the recorded temperatures in the one-dimensional heat transfer models using ABAQUS. In addition, thermogravimetric analysis and X-ray diffraction were used to characterise the phase transformations of samples under high temperature. All the experimental results of CNF-LCC and LCC were compared with traditional foam concrete, NWC and LWAC from published results to evaluate their performance under high temperature. The effect of foam bubbles and CNFs on these thermal properties was also investigated.

4.2 Experimental Programme

4.2.1 Laser Flash

Thermal diffusivity is an inherent thermal property of a material and is related to thermal conductivity. The thermal diffusivity of a sample was obtained using TA Discovery Laser Flash DLF1200. The samples were cast as discs with a diameter of 12.5 mm and a thickness of 2 mm. A thin layer of graphite coating was applied over the samples to ensure that the energy pulse was absorbed by the samples. They were heated from ambient to 800 °C with a heating rate of 10 °C/min and diffusivity was measured at an interval of 100 °C.

4.2.2 Modulated Differential Scanning Calorimetry (MDSC)

Specific heat capacity of the samples was measured using TA Discovery DSC. The samples were grounded into powder using mortar and pestle and 4 to 5 mg of powder was placed in Tzero hermetic pans. The samples were heated from 10 °C to 400 °C at a heating rate of 10 °C/min. Modulated differential scanning calorimetry was conducted with an amplitude of 1.592 °C at a period of 60 seconds.

4.2.3 Thermomechanical analysis (TMA)

Thermal strain of the samples was measured using TA TMA Q400. The samples were cast as cylinders with a height and diameter of 6 and 5 mm, respectively. A force of

0.01 N was applied to ensure contact between the probe and the sample surface and a temperature ramp of 20 °C/min was imposed from ambient to 800 °C.

4.2.4 Thermogravimetric analysis (TGA)

The mass loss of samples with increasing temperature was measured by means of thermogravimetric analysis (TGA) with a heating rate of 10 °C/min from 20 to 800°C.

4.2.5 X-Ray Diffraction (XRD)

The crystal phases formed during hydration of cement and after exposure to heating could be identified using X-Ray diffraction. The samples were exposed to 800 °C.

The samples were kept in the furnace for 3 hours to ensure that the temperature distribution within the sample was constant. The samples were then removed and grounded into powder using mortar and pestle to eliminate any preferred orientation effects in the results. The X-Ray diffraction was performed with Bruker D8 Advance with Cu-K α radiation and 40 kV current. A step size of 0.02 ° was used from 5 to 80°.

4.2.6 One-dimensional heat transfer tests on CNF-LCC/LCC blocks

To validate the measured specific heat and thermal conductivity for CNF-LCC/LCC, one-dimensional heat transfer tests were conducted and the experimental temperature profiles in CNF-LCC/LCC samples were compared with the predicted results obtained from ABAQUS V6.14 where the measured thermal properties (thermal diffusivity, specific heat and thermal conductivity) were used as input data. The

dimensions of the CNF-LCC / LCC blocks were $150 \text{ mm} \times 150 \text{ mm} \times 50 \text{ mm}$ (length $\times$ width $\times$ thickness). Each block was supported with gypsum boards and placed at the opening of an electric furnace as shown in Figure 4.1. The setup ensured that only one face of the block was subjected to the furnace temperature and one-dimensional heat transfer could be achieved. The electric furnace could simulate the time-temperature relationship of ISO standard fire curve (BSI, 2012a) as shown in Figure 4.2. Two type-K thermocouples were placed on the exposed side and half of the thickness at the centre of the block sample, respectively, to record the temperature development as shown in Figure 4.1.

Figure 4.1 Schematic diagram of furnace and test samples

Figure 4.2 Time-temperature relationship of ISO standard fire curve and furnace gas

4.3 Results and discussion

4.3.1 Phase transformations under high temperature

The phase transformations of ceUHPC, CNF-LCC and LCC during heating can be studied using thermogravimetric analysis (TGA) and X-Ray diffraction (XRD) and the results are shown in Figure 4.3 and Figure 4.4, respectively. Under high temperature, mass loss in concrete attributed to evaporation of free water, dehydration and decomposition reactions that influence thermal properties are discussed in this study. Similar mass loss curves of ceUHPC, CNF-LCC and LCC in Figure 4.3 indicated that adding foam bubbles or CNFs has negligible effect on mass loss during heating. The first step at about 150 °C corresponded to the removal of free water and dehydration of ettringite (Alonso and Fernandez, 2004, Khoury, 2008). In addition,

the loss of bound water and dehydration of C-S-H started from about 100 °C (Pimienta et al., 2017). Between 150 and 650 °C, gradual and slow mass loss was observed. In this process, most of the bound water was removed at 250 °C and carbonation of portlandite (Ca(OH)_2) between 200 to 500 °C resulted in an increased amount of calcite (CaCO_3) (Piasta et al., 1984). The portlandite which does not undergo carbonation will decompose at 450 ~ 550 °C to form lime (CaO) (Piasta et al., 1984, Alonso and Fernandez, 2004). Dehydration of C-S-H accelerated from 200 °C and the dehydrated product was named as new nesosilicate (Alonso and Fernandez, 2004, Piasta et al., 1984). The second mass loss from 650 to about 750 °C was attributed to the decomposition of CaCO_3 and CO_2 gas in the pore structure (Alonso and Fernandez, 2004, Pimienta et al., 2017). When comparing the XRD results obtained at 25 and 800 °C as shown in Figure 4.4, typical reflection peaks of Ca(OH)_2 and CaCO_3 at room temperature disappeared after heating to 800 °C and only CaO was found. Furthermore, by 800 °C, all the C-S-H was transformed to new nesosilicate by dehydration (Alonso and Fernandez, 2004, Piasta et al., 1984) and it can be confirmed by the increased intensity of the peaks related to C_2S and C_3S from 25 to 800 °C in Figure 4.4 because the new nesosilicate is a crystalline phase, similar to the structure of C_2S .

Figure 4.3 Mass loss of ceUHPC, CNF-LCC and LCC from 23 to 800 °C

Figure 4.4 X-ray diffractograms of samples at room temperature and 800°C (a) CNF-LCC; (b) LCC

4.3.2 Thermal diffusivity

Thermal diffusivity represents the thermal inertia of a material and can be defined as the ability of a material to conduct heat relative to the heat stored per unit volume. A material with high diffusivity will attain thermal equilibrium faster than materials with low thermal diffusivity. The measured thermal diffusivity of ceUHPC, CNF-LCC and LCC are shown in Figure 4.5 as a function of temperature. It was found that CNF-LCC & LCC have similar values of thermal diffusivity from room temperature to 800 °C. Thermal diffusivity of CNF-LCC & LCC decreases as temperature increases up to 600 °C and subsequently increased slightly. However, the thermal diffusivity of ceUHPC is higher than CNF-LCC & LCC and keeps decreasing during

the heating process. At room temperature, the thermal diffusivity of CNF-LCC & LCC is 0.52 and 0.58 mm²/s respectively and they are about 25.7% averagely lower than ceUHPC (0.74 mm²/s) because of the introduction of foam bubbles. Thermal diffusivity of CNF-LCC & LCC is much lower than NWC which ranges from 1.1 to 1.2 mm²/s depending on the hydration degree of cement (De Schutter and Taerwe, 1995). Mydin et al. (2012) measured the thermal diffusivity of foam concrete with 1400 kg/m³ is 0.53 to 0.66 mm²/s with different mix designs, which are close to the results presented in this study although CNF-LCC & LCC have higher density. The temperature-dependent thermal diffusivity values of NWC and LWAC from Felicetti (2007) are also plotted for comparison in Figure 4.5. The diffusivity of CNF-LCC & LCC is always lower than NWC and similar to LWAC while CNFs do not show obvious effect on thermal diffusivity.

Figure 4.5 Thermal diffusivity of ceUHPC, CNF-LCC, LCC, NWC and LWAC from 23 to 800 °C

4.3.3 Specific heat

Specific heat or heat capacity is the amount of heat per unit mass required to raise the temperature by 1 °C. The experimental results of temperature-dependent specific heat of ceUHPC, CNF-LCC and LCC are shown in Figure 4.6. It was found that the curves of these three samples are very close and introducing foam bubbles or CNFs has minimal influence. The specific heat value of all samples increases with increasing temperature and reaches a peak value at about 150 °C, followed by a decrease to about 300 °C and remains constant over the remaining temperature range. This performance is similar to the developed constitutive relationship between specific heat and temperature of NWC and foam concrete (Othuman and Wang, 2011). At ambient temperature, the values of specific heat of ceUHPC, CNF-LCC and LCC are very close, between 753 to 797 J/(kg·K). According to the mixture law, the specific heat of concrete is equal to the sum of each component specific heat multiplied by the corresponding weight fraction (Wang, 1995). The introduced air bubbles only increase the volume and their weight can be neglected; hence, the weight fraction of cement, sand, and water in ceUHPC, CNF-LCC, and LCC are the same and consequently, they have similar values of specific heat. The common range of specific heat of NWC is 840 to 1170 J/(kg·K) (Neville, 1995). The specific heat of LWAC with density 1400 to 1900 kg/m³ is between 837 and 1074 J/(kg·K) (Real et al., 2016). Kodur and Sultan (2003) measured the specific heat of high-strength concrete (HSC)

with different types of coarse aggregate and the value was around 760 J/(kg·K). Specific heat is highly dependent on moisture content and increases considerably with higher water-cement ratio because the specific heat of water is about 4180 J/(kg·K) while the values for cement and aggregate are about 920 and 800 J/(kg·K), respectively (Othuman and Wang, 2011, Wadsö et al., 2012, Kodur, 2014). The specific heat of ceUHPC, CNF-LCC, and LCC at room temperature is comparable with HSC but lower than NWC and LWAC, which is attributed to the lower water cement ratio and consequently lower moisture content. With the temperature increasing, the specific heat values of ceUHPC, CNF-LCC, and LCC increase substantially up to about 150 °C which is caused by the evaporation of free water and loss of bound water (Lie, 1972, Khoury, 2008). In this temperature region, most of the heat energy is absorbed to remove free water from concrete and a small amount is provided to increase its temperature. The maximum values for ceUHPC, CNF-LCC, and LCC range from 1322 to 1400 J/(kg·K) are very close. The additional heat required to drive off water is proportional to the water content in concrete (Nguyen et al., 2009), which is critical to the maximum specific heat at a temperature under 200 °C (Pimienta et al., 2017). The maximum specific heat value of the three samples in Figure 4.6 is similar with HSC from Kodur and Sultan (2003) but lower than foam concrete in Othuman and Wang (2011) because the water-cement ratio of the former is similar as in this study but that of the latter is higher.

Figure 4.6 Specific heat of ceUHPC, CNF-LCC and LCC from 23 to 800 °C

4.3.4 Thermal conductivity

Thermal conductivity represents the intrinsic ability of a material to conduct heat. It can be defined as the rate of heat flow through a body of unit thickness and unit area with a unit temperature difference between the two surfaces. In this study, thermal conductivity λ (W/(m·K)) is given by the product of thermal diffusivity α_p (m²/s), specific heat c_p (J/(kg·K)), and density ρ_d (kg/m³) as expressed in the following equation:

$$\lambda = \alpha_p \cdot c_p \cdot \rho_d \quad (4.1)$$

The thermal conductivity of ceUHPC, CNF-LCC, and LCC can be calculated by Equation 4.1 and the results are shown in Figure 4.7. It can be seen that ceUHPC always exhibit higher thermal conductivity than CNF-LCC and LCC over the entire

temperature range. Introducing foam bubbles can effectively reduce thermal conductivity as air has low thermal conductivity. However, CNFs have minimal influence on thermal conductivity when comparing the curves of CNF-LCC and LCC although thermal conductivity of CNFs can be as high as about 1400 W/mK (Bauer et al., 2016). This may be attributed to low volume percent of CNFs in the matrix that cannot form an effective network to provide a proper pathway for heat conduction. Meanwhile, the interfacial thermal resistance between CNFs/CNTs and surrounding matrix may also weaken the heat conduction, analogous to the condition of CNFs used in the polymer (Han and Fina, 2011). At room temperature, the thermal conductivity of CNF-LCC and LCC is 0.62 W/mK and 0.66 W/mK, respectively. Comparing with the published data in Table 2.2, the thermal conductivity of CNF-LCC & LCC is much lower than NWC, which is expected, but higher than traditional foam concrete with similar density. It is attributed to the low water cement ratio and high packing density of solid material in the mix design of CNF-LCC & LCC resulting in lower porosity in the base mix (Real et al., 2016). The thermal conductivity values of eUHPC, CNF-LCC, and LCC increase with increasing temperature and reach a peak value at about 150 °C. This phenomenon is because the samples in this study were not dried before testing and the evaporation of water, associated with the loss of latent heat of vaporisation, increases the specific heat substantially, resulting in an increase of thermal conductivity based on Equation 4.1 (Nguyen et al., 2009). It can also be supported by comparing the curves of moist and

dried limestone concrete in Figure 4.7 (Schneider, 1988). Between 150 and 600 °C, the thermal conductivity values of ceUHPC, CNF-LCC and LCC demonstrate decreasing behaviour which is similar to NWC. The reduction of thermal conductivity is attributed to the formation of microcracks under high temperature leading to increased porosity in concrete (Kizilkanat et al., 2013). After 600 °C, CNF-LCC and LCC show an increasing trend of thermal conductivity, which is different from ceUHPC and NWC. This can be explained by the fact that radiation plays a more important role than heat conduction in foam concrete under high temperature, increasing its effective thermal conductivity (Othuman and Wang, 2011).

Figure 4.7 Thermal conductivity of ceUHPC, CNF-LCC, LCC, NWC from 23 to 800 °C

Real et al. (2016) conducted experimental work on thermal conductivity of LWAC with a wide range of mix design, compressive strength and density, covering the most

often used LWAC for structural elements. They proposed a factor k , a ratio between structural efficiency and thermal conductivity (Equation 4.2), to evaluate the performance of NWC and LWAC, where f_{cm} is compressive strength (MPa), ρ is density (kg/m^3) and λ is thermal conductivity (W/mK). As shown in Table 4.1, the k value of CNF-LCC is higher than all the LWAC and NWC reported in their research. Therefore, CNF-LCC can be considered as an excellent thermal insulation structural material for its low thermal conductivity and comparable compressive strength.

$$k = f_{cm}/(\rho\lambda) \quad (4.2)$$

Table 4.1 k value of CNF-LCC, LCC, NWC and LWAC

Concrete	Density (kg/m^3)	Compressive strength (MPa)	Thermal conductivity (W/mK)	k value
NWC	2076 ~ 2324	20.4 ~ 84.2	1.35 ~ 2.00	0.006 ~ 0.019
LWAC	1441 ~ 1883	14.8 ~ 66.8	0.7 ~ 1.36	0.011 ~ 0.029
CNF-LCC	1500	30.2	0.62	0.032
LCC	1500	26.8	0.66	0.027

4.3.5 One-dimensional heat transfer tests on CNF-LCC/LCC blocks

The measured thermal properties of CNF-LCC and LCC in Section 4.3 - 4.5 can be validated with experimental results obtained from the one-dimensional heat transfer tests in Section 4.6. Heat transfer can be modelled using simulation software

ABAQUS V6.14. Validation of the numerical model with ABAQUS software has been reported in Ng et al. (2017).

Heat transfer analysis in ABAQUS can account for radiation and convection using emissivity and coefficient of convection. Although these two parameters are not investigated in this study, they are typically well-defined in literature. Conduction of heat through a concrete sample is governed mainly by thermal conductivity value. In view of this, the measured surface temperature of the samples in one-dimensional heat transfer tests will be used as input in the simulation. This can eliminate the uncertainty in thermal boundary conditions on the exposed side as reported by Othuman and Wang (2011).

The rear face of the concrete sample was exposed to the ambient environment that served as a heat sink. Convection and radiation from the sample to the ambient environment are accounted for in the model. Although one-dimensional heat transfer governs the experimental setup shown in Figure 4.1, there is heat loss from the concrete sample to the supporting gypsum board. To improve the accuracy of the predictions, gypsum boards that were used to support the concrete sample are also included in the model and heat transfer from the edges of the concrete sample to the gypsum boards is simulated as perfect contact. In addition, heat transfer from the heated gas in the furnace to the gypsum boards, through the gypsum board, and to the ambient environment is also simulated.

The time-temperature profile at 25, 30, and 35mm from the heated surface are extracted from the heat transfer model and compared with the experimental time-temperature profile. The time-temperature profile at these three depths is chosen as the thermocouple was embedded at the core of the concrete sample (approximately 25mm from the heating surface). It was possible for the thermocouple to displace slightly after casting and was not located exactly at 25 mm in the concrete sample.

Figure 4.8 Experimental and analytical time-temperature development profile of CNF-LCC and LCC

As seen in Figure 4.8, the deviations between the numerical predictions and the experimental results for CNF-LCC and LCC are observed after approximately 80 min. The change in this temperature history may be due to an increase in the ambient temperature, which was not accounted for in the numerical model as it may reduce the rate at which heat is lost from the sample. The slight plateau in temperature at 100

$^{\circ}\text{C}$ is also not apparent in the simulated results. This suggests that the evaporation of water is not well-captured in numerical modelling. This is expected as the sample is ground to 5 mg of fine powder for the MDSC measurement. Free water in capillary pores that contributes to this behaviour is reduced as the concrete samples are ground to powder. Nonetheless, the simulated results for CNF-LCC and LCC are in fair agreement with the experimental results and supports the validity of the measured thermal properties and the derivation of thermal conductivity in Section 4.3.4.

4.3.6 Thermal expansion

The thermal strains of ceUHPC, CNF-LCC, LCC and cement paste (with the same water to cement ratio and SP content) are given in Figure 4.9 for temperature between 25 and 800 $^{\circ}\text{C}$. All the specimens showed slight expansion before 75 $^{\circ}\text{C}$ because the evaporation of water increased the internal pressure in the samples. The XRD results of CNF-LCC and LCC in Figure 4.4 indicate the existence of unhydrated cement (C_2S and C_3S) at room temperature which expands with increasing temperature (Piasta, 1984). After 75 $^{\circ}\text{C}$, all the samples showed shrinkage behaviour caused by loss of free water and dehydration of hydrated paste. Cement paste has the maximum shrinkage values and it contracts up to about 670 $^{\circ}\text{C}$. However, the shrinkage of ceUHPC, CNF-LCC and LCC was much smaller than cement paste because quartz sands in these samples will expand during heating (Cruz and Gillen, 1980) and shrinkage is compensated. ceUHPC contracted up to about 450 $^{\circ}\text{C}$ whereas CNF-

LCC & LCC contracted up to about 200°C and their shrinkage was smaller than ceUHPC. The content of cement paste in CNF-LCC & LCC was lower than ceUHPC due to the introduction of foam bubbles which lower the content of water and hydrated products. The first derivation of the thermal strain curves gives the phases vibration of the samples as shown in Figure 4.10. The peaks between 100 to 200 °C correspond to shrinkage that is mainly caused by evaporation of free and bound water from capillary pores (Piasta, 1984). Based on the capillary tension theory, unsaturated water in capillary pores forms a meniscus surface and the water surface tension will produce a negative water pressure in capillary pores resulting in shrinkage of concrete (Shimomura and Maekawa, 1997). It was reported that the nature of foam agent is similar to shrinkage-reduction admixture that can work as a surfactant to reduce water-surface tension and thus the shrinkage of CNF-LCC & LCC is lower than ceUHPC. When comparing the thermal shrinkage between CNF-LCC and LCC, it was found that CNFs can reduce shrinkage. This is attributed to the finer capillary pore (especially 10~50 nm size which controls shrinkage (Mindess et al., 2003)) filled by CNFs with diameters of a similar range. Between 550 and 590 °C, thermal strains of ceUHPC, CNF-LCC and LCC increase suddenly, which can also be reflected by the peaks in Figure 4.10 at the same temperature range. The expansion is because quartz sands in the matrix undergo a transformation from α to β form at 573 °C (Cruz and Gillen, 1980). After this period, the thermal strains of all samples do not vary up to about 670 °C. The thermal strain of CNF-LCC & LCC remained nearly constant

to the end but ceUHPC and cement paste started to increase until 800 °C. The expansion in ceUHPC and cement paste was because CO₂ gas generated from the decomposition of calcite at 650 °C, as discussed in Section 4.3.1, increased internal pressure in the pores. However, the thermal strain curves of CNF-LCC & LCC show no increase in this period because the porous structure of them provides sufficient space to reduce the internal pressure generated by CO₂ gas.

The thermal strains of different types of NWC and LWAC are also plotted in Figure 4.9. All the curves expand with increasing temperature because the coarse aggregates used in their matrix account for large volume proportion and have a high coefficient of thermal expansion during heating (Cruz and Gillen, 1980). LWAC shows lower thermal expansion than NWC because the production of lightweight coarse aggregates is under high temperature and they have inherent fire stability with low thermal expansion (ACI Committee 213, 2003). Both CNF-LCC and LCC exhibit more stable thermal strain and lower expansion than NWC and LWAC. When the structure is exposed to different temperatures, smaller expansion will result in lower thermal stress that eliminates local damage and reduces the tendency to bending of structure elements (Neville, 1995).

Figure 4.9 Thermal strain of ceUHPC, CNF-LCC, LCC, cement paste, NWC and LWAC from 23 to 800 $^{\circ}\text{C}$

Figure 4.10 First derivate of thermal strain of ceUHPC, CNF-LCC, LCC and cement paste

4.4 Summary

A series of experiments showed that CNF-LCC had reasonably good thermal insulation properties and low thermal expansion for fire resistance. The conclusions of CHAPTER 4 are as follows:

1. Adding CNFs had no effect on the thermal diffusivity, specific heat and thermal conductivity, which can be reflected by the TGA and XRD results. Introducing foam bubbles can effectively reduce thermal diffusivity and thermal conductivity but had no influence on specific heat capacity.
2. The thermal diffusivity of CNF-LCC was always lower than NWC and was similar to LWAC. The specific heat of CNF-LCC at room temperature and during water evaporation period (maximum value) was lower than NWC and LWAC but comparable to HSC because CNF-LCC utilised low water-cement ratio (similar to that of HSC).
3. During heating, thermal conductivity of CNF-LCC was always lower than NWC. At room temperature, the thermal conductivity of CNF-LCC was also lower than LWAC but slightly higher than traditional foam concrete with the same density. This was attributed to its low water-cement ratio and high packing density in the mix design resulting in low porosity of the base mix (ceUHPC). CNF-LCC also showed

better comprehensive performance than NWC and LWAC when considering the ratio between structural efficiency and thermal conductivity.

4. Good agreement between one-dimensional heat transfer test results and numerical results from ABAQUS verified the measured thermal diffusivity, specific heat and estimated thermal conductivity under high temperature.

5. Introducing foam bubbles can reduce thermal shrinkage during water evaporation phase due to reduced cement paste content and shrinkage-reducing nature of the foam agent. Thermal shrinkage can also be reduced by adding CNFs because they can further reduce finer capillary porosity due to nanoscale dimension and high specific area. The porous structure of CNF-LCC can reduce internal pressure caused by the generation of CO₂ gas and eliminate thermal expansion after 670 °C. The more stable and smaller thermal expansion, as well as better thermal insulation than NWC and LWAC during heating, showed that CNF-LCC performs well in fire conditions.

CHAPTER 3 and CHAPTER 4 investigate the short-term material properties of CNF-LCC. However, the long-term material properties such as durability, shrinkage and creep of concrete have to be considered and allowed for in design because they ensure safety of structures under long-term service loading. The next chapter focuses on the long-term properties of CNF-LCC.

CHAPTER 5 DURABILITY, SHRINKAGE AND CREEP OF CNF-LCC

5.1 Introduction

The engineering properties of CNF-LCC investigated in previous chapters can be classified as the short-term properties of concrete. However, the long-term or time-dependent properties are also very important for new building material to contain both an aesthetically pleasing view and long-lasting service. The long-term properties of concrete include shrinkage, creep and durability. The shrinkage and creep of concrete are caused by the same internal process which involves the movement of moisture. The shrinkage of concrete results from loss of moisture due to hydration of unhydrated cement (autogenous shrinkage) or surrounding unsaturated environment (drying shrinkage). The creep of concrete, however, is caused by the movement of moisture from one location to another within the concrete because of the sustained stress. The durability characteristics can be evaluated by the porosity and permeability of concrete. Furthermore, the shrinkage, creep and durability (permeability) are controlled by different sizes of pores at the nano- or micro-level as shown in Table 5.1. In this chapter, the durability of CNF-LCC is evaluated by water penetration depth tests and the results are converted to coefficient permeability. The shrinkage and creep of CNF-LCC were measured for a period of one year and mercury intrusion porosimetry tests were conducted to investigate the distribution of

nano-size pores. All the long-term properties of CNF-LCC were compared with those of NWC or LWAC at a similar testing environment and the effect of CNFs was studied by comparing with the control sample LCC.

Table 5.1 Properties of concrete controlled by different sizes of pores (Neville, 1995)

Type of pores	Description	Size	Research methods	Properties of concrete
Interlayer space	Structural	< 0.5 nm	Absorption/thermal	Shrinkage and creep (<11% RH)
Gel pores	Micropores	0.5 ~ 2.5 nm	Absorption/MIP/IS	Shrinkage and creep (11 ~ 35% RH)
	Small	2.5 ~ 10 nm		Shrinkage (to 50% RH)
Capillary pores	Medium	10 ~ 50 nm	SEM/OM	Permeability, shrinkage and strength (high RH)
	Large	50 nm ~ 10 μ m	SEM	Permeability and strength
Other features	ITZ	20 ~ 50 μ m	SEM/OM	Permeability and strength
	Microcracks	50 ~ 200 μ m		Permeability and strength

Noting: MIP is mercury intrusion porosimetry; IS is independence spectroscopy; SEM is scanning electron microscopy; OM is optical microscopy.

5.2 Experimental Programme

5.2.1 Water penetration depth

The test was performed by clamping a 150 mm cube between two flanges with special circular gaskets. Under a controlled pressure of 500 ± 50 kPa for 72 ± 2 h, water was applied to the surface of the concrete specimen. Penetration of water was measured after the testing period by breaking apart the specimen. The test complied with BS EN 12390-8 (2009a) and three samples were prepared in each case. To correlate the

direct measurements of water flowing through a concrete specimen with the coefficient of permeability, an expression was developed to relate the depth of water penetration with the coefficient of permeability, equivalent to that used in Darcy's law (Valenta, 1969).

$$K = e^2 v / (2 h_w t_p) \quad (5.1)$$

where K is the coefficient of permeability (m/s), e is the depth of penetration (m), v is the fraction of concrete volume occupied by pores which can be assumed as 0.02 (Ho et al., 2015), h_w is the hydraulic head (m), and t_p is the test duration (s).

5.2.2 Shrinkage and creep

Φ150×300 mm cylinder specimens were cast for the determination of the autogenous shrinkage, total shrinkage and creep of CNF-LCC and LCC. All the samples were demoulded 24 hours after casting, and the curing, as well as testing, were conducted at a controlled laboratory condition with 28 °C temperature and 75% relative humidity (normal weather condition in Singapore). The autogenous shrinkage specimens were sealed by adhesive aluminum tape immediately after demoulding. A modified ASTM C426 (2010) method was used to measure autogenous shrinkage by using demountable mechanical (Demec) gauges. The Demec gauges were glued by a five-minute epoxy resin with a 100 mm gauge length on the opposite sides of cylinder surface for shrinkage measurements. The total shrinkage and creep specimens were

directly fixed with Demec gauges using the same procedure after demoulding. A digital indicator with 0.001mm accuracy was used to measure the change in length. Measurements of autogenous and total shrinkage started from the day of fixing of Demec gauges on the specimens up to 365 days. It was assumed that drying shrinkage is the difference between total and autogenous shrinkage. The creep specimens were cured at the laboratory air condition up to 28 days. The creep test process was in accordance with BS ISO 1920-9 (2009b). At the age of 28 days, the creep specimens were loaded on a hydraulically controlled creep frame to a stress level of one-third of cylinder compressive strength and this applied load was sustained for a duration of 365 days. The creep strain has a direct linear relationship with the applied stress when the ratio between the applied stress and concrete strength does not exceed an upper limit (BSI, 2004, Neville, 1995). This upper limit represents the development of severe microcracking in concrete and the relationship between creep and applied stress is non-linear after exceeding the upper limit (Neville, 1995). The limit in concrete commonly ranges from 0.4 to 0.6, but occasionally as low as 0.3 or as high as 0.75; the latter value is employed to high strength concrete (SmadiI and Slate, 1989). For mortar material, the limit is between 0.80 and 0.85 (Ross, 1958). In EC 2 (BSI, 2004), for structural concrete, the limit of linear creep is 0.45 times compressive strength, beyond which, nonlinear creep prevails. The applied load intensity was maintained within a 2% variation by adjusting the load with the hydraulic pump to

compensate for the load reduction over time resulting from shrinkage and stress relaxation of the specimens (BSI, 2009b).

5.2.3 Mercury intrusion porosimetry (MIP) test

The total porosity and pore size distribution of cement-based materials can be evaluated by MIP test which is based on the method of intruding mercury into the pore structure under precisely controlled pressure. In this study, the MIP test was conducted on Autopore IV 9510 that can provide faster and more accurate measurements for pore size ranging from 3 nm to 1000 μm . The small block samples obtained from the cylinder samples after compressive tests were used for the MIP tests. The specimens were oven-dried at 60 $^{\circ}\text{C}$ for 48 hours followed by vacuum dry for 48 hours. The high pressure up to 60,000 psi was used to pressure the mercury into the sample pores with a minimum size of about 3 nm.

5.3 Results and discussion

5.3.1 Water penetration depth

The test results for water penetration test and converted values of the water permeability coefficient are presented in Figure 5.1. The results of NWC in Figure 5.1 are from Ho et al. (2015) in which the sample size, curing age and testing procedure are the same as those in this study. It was found that the water penetration depth and water permeability coefficient of both LCC and CNF-LCC are extremely

lower than NWC. This is attributed to the low capillary porosity in the UHPC/ceUHPC base mix and the unconnected foam bubbles. Furthermore, CNF-LCC significantly lowers the water penetration depth and permeability level by 45.8% and 70.6%, respectively, compared to the LCC mix. According to Mindess et al. (2003), water in capillary pore size from 10 nm to 10 μm is sensitive to permeability and the range of CNFs diameters used in this study falls into this range. Based on the micro-structural SEM imaging, the CNFs provided good coverage with a different range of diameters in the matrix. Figure 5.2 (a) shows the SEM analysis for CNF-LCC mix; CNFs made the overall CNF-LCC matrix denser than LCC. The homogeneous dispersion of CNFs with a different range of diameters was well bonded with calcium hydroxide and C-S-H gel (Chen et al., 2016). This finding showed that CNFs enhanced and produced a denser matrix to eliminate capillary pores which contain evaporable bulk water (Mindess et al., 2003). Overall, CNF contributed to a denser microstructural formation with nano-sized crystallites Figure 5.2 (a)). On the other hand, Figure 5.2 (b) shows the LCC microstructure (with larger pore size) is mainly formed by the calcium hydroxide large plate and platy crystals. Thus, larger capillary pores lead to a higher water permeability for LCC.

Figure 5.1 Water penetration and water permeability coefficient of LCC, CNF-LCC, and NWC

(a)

(b)

Figure 5.2 SEM images of (a) CNF-LCC; (b) LCC

5.3.2 Shrinkage behaviour

Figure 5.3 shows the experimental results of LCC and CNF-LCC for one-year shrinkage. The predicted results of NWC and LWAC were also plotted in Figure 5.3 according to the equations provided by EC 2 (2004). It should be mentioned that the equations in EC 2 consider the effect of many factors on the shrinkage of NWC and LWAC including sample size, compressive strength, relative humidity and cement type.

Figure 5.3 Different shrinkage strain of LCC, CNF-LCC, NWC, and LWAC (AS: autogenous shrinkage; DS: dry shrinkage; TS: total shrinkage)

Autogenous shrinkage of LCC & CNF-LCC is higher than NWC & LWAC and still keeps increasing after 28 days. However, drying shrinkage of LCC & CNF-LCC is

much lower than NWC & LWAC, which results in lower total shrinkage. These results are due to the low water/cement ratio in the base mix of LCC & CNF-LCC. On the one hand, a low water/cement ratio generates a low degree of hydration and a higher amount of unhydrated cement will hydrate with water in a long time, resulting in higher and ever-increasing autogenous shrinkage (Wu et al., 2017). On the other hand, a low water/cement ratio will reduce water loss due to evaporation and densify the base mix with a lower amount of capillary pores (Tam et al., 2012). Thus, drying shrinkage of LCC & CNF-LCC was lower than NWC & LWAC. Shrinkage is mainly caused by water loss from capillary pores. Based on capillary tension theory, unsaturated water in capillary pores will form a meniscus surface and the water surface tension will produce a negative water pressure in capillary pores resulting in shrinkage of concrete (Shimomura and Maekawa, 1997). Therefore, low water/cement ratio and high packing density of UHPC based LCC & CNF-LCC have a lower amount of capillary pores and consequently lower shrinkage than NWC & LWAC. Besides, it was reported that the nature of foam agent is similar to a surfactant which is the same with the shrinkage-reduced admixture to reduce water-surface tension and thus shrinkage can be further reduced (Nambiar and Ramamurthy, 2009).

When comparing the results of LCC and CNF-LCC, it was found that CNFs can effectively reduce autogenous and drying shrinkage simultaneously. It was reported that the capillary pores with 10~50 nm size dominate the shrinkage in concrete (Mindess et al., 2003). Ziembicka (1977) and Georgiades et al. (1991) related the

shrinkage of cellular concrete and autoclaved aerated concrete to the volume and specific area of micropores with diameter 15 ~ 125 nm and 4 ~ 40 nm, respectively. MIP testing results in Figure 5.4 show the relationship between cumulative intrusion volume per unit specimen mass and pore diameter, and differential pore size distribution of ceUHPC and UHPC base mix. Total intrusion (corresponding to the point of smallest pore size in the cumulative intrusion curves) of ceUHPC is about 29% lower than that of UHPC, indicating lower porosity by addition of CNFs. Comparing the differential intrusion curves in Figure 5.4, the critical pore diameter for both ceUHPC and UHPC is about 32 nm and introducing CNFs can significantly reduce the volume of this size of capillary pore. In summary, the range of capillary pores that dominates shrinkage of foam concrete varies from 4 to 125 nm based on (Mindess et al., 2003, Ziembicka, 1977, Georgiades et al., 1991) and the pore volume in this dominant pore range can be estimated by the area under the differential intrusion curve (Aligizaki, 2005). The calculated area under differential curves is shown in Figure 5.4 and ceUHPC has about 31% lower pore volume than UHPC by adding CNFs. The MIP testing results confirmed that the essential capillary pores in the UHPC base mix are filled in by CNFs with diameters of a similar range (see Figure 3.2 (a)); the capillary porosity is further reduced and shrinkage can be decreased, which coincides with the findings in (Konsta-Gdoutos et al., 2010b)

Figure 5.4 MIP test results of UHPC and ceUHPC

5.3.3 Creep behaviour

5.3.3.1 The effect of CNFs

Creep strain was determined by Demec measurements taken on the creep cylinder samples described in Section 5.2.2 by subtracting total shrinkage strain developed from 28 days which is the start of applied load for creep. Figure 5.5 shows the average creep strain of CNF-LCC and LCC with $1500 \pm 50 \text{ kg/m}^3$ density over time. It was found that the creep-time curves of both CNF-LCC and LCC show an obvious reduction in the slopes, i.e., gradually decreased creep rate over time. However, CNF-LCC exhibits less overall creep strain than that of LCC. The one-year creep strain of LCC is $788 \mu\epsilon$ which is 18.3% higher than the corresponding creep strain of CNF-

LCC with 644 $\mu\epsilon$. With a wide range of data, creep of concrete will be generally decreased with increased compressive strength, decreased shrinkage or decreased permeability (Neville, 1995, L'HERMITE, 1960, Lopez, 2005). By adding CNFs, the compressive strength, durability and shrinkage resistance of CNF-LCC is better than LCC as presented in Section 3.3.1, 5.3.1 and 5.3.2, respectively. However, this indirect evidence cannot explain the effect of CNFs on creep from nanoscopic or microscopic perspective. Short-term creep is caused by micro-diffusion of absorbed or intercrystalline water from gel pores into capillary pores like an internal seepage (Neville, 1995, Hewlett and Liska, 2019). Similar to the effect of CNFs on shrinkage as explained in Section 5.3.2 CNFs utilised in this study can effectively reduce capillary porosity in the UHPC base mix and decrease pathway for movement of gel water to capillary pores, which results in a creep reducing. On the other hand, long-term creep originates from the sliding of C-S-H that leads to gradually reduced creep rate as a result of aging (Neville, 1995, Hewlett and Liska, 2019). Research studies using nanoindentation techniques indicated that sliding results in a local increase in packing density until a limit state of three compositionally similar but structurally distinct forms of C-S-H: low density C-S-H, high density C-S-H and ultra-high density C-S-H (Hewlett and Liska, 2019, Nguyen et al., 2014). The creep of ultra-high density C-S-H and high density C-S-H is lower than low density C-S-H because their higher packing density makes them difficult to be further compacted under sustained stress. With the help of statistical nanoindentation method, Barbhuiya and

Chow (2017) observed that incorporating CNFs effectively increased the amount of high density C-S-H at the cost of low density C-S-H, which consequently explains the decreased creep behaviour by adding CNFs as shown in Figure 5.5.

Figure 5.5 Creep strain of CNF-LCC and LCC with time

5.3.3.2 Comparison with NWC and LWC from literature

The measured creep results of CNF-LCC were also compared with the experimental results of NWC and LWC published in the literature as summarised in Table 5.2. Due to different applied creep loads in different tests, it is more meaningful to compare the specific creep (creep strain divided by applied stress) of CNF-LCC and other concrete from publication because there is a direct linear proportionality between creep strain and applied stress when the creep stress is less than 0.4 of compressive strength. The comparison results are shown in Figure 5.6 to Figure 5.8. Although there is an absence of coarse aggregates to restrain the creep, CNF-LCC shows

comparatively lower specific creep compared with NWC of similar compressive strength reported by Best and Polivka (1959) and Van der Wegen and Bijen (1985) as shown in Figure 5.6. However, the specific creep of CNF-LCC is greater than the results of NWC obtained from Tang et al. (2014) and Wendling et al. (2018), which may be attributed to the lower compressive strength of CNF-LCC. Similar results are obtained when CNF-LCC is compared with LWAC. Figure 5.7 shows that CNF-LCC exhibits comparable or lower specific creep than LWAC of similar strength measured by Best and Polivka (1959) and Van der Wegen and Bijen (1985). Lopez et al. (2004) and Wendling et al. (2018) studied the creep behaviour of high performance lightweight concrete (HPLC) with 67.3 MPa compressive strength and lightweight self-compacting concrete (LWSCC) with 46 MPa compressive strength, respectively. Both of them present lower specific than CNF-LCC, which might be due to the difference in mix composition and compressive strength. It is very difficult to compare the creep of CNF-LCC with traditional foam concrete because there is very scarce data in this area. Polystyrene aggregate concrete (PAC) could be a good comparison because the polystyrene aggregate (PA) is made from small lightweight Styrofoam or expanded polystyrene balls that have a similar spherical shape with foam bubbles. Tang et al. (2014) investigated the creep of PAC with density varying from 1410 to 2120 kg/m³ and corresponding compressive strength from 9.3 to 32 MPa. Their experimental results are compared with CNF-LCC as shown in Figure 5.8. It was found that CNF-LCC has similar specific creep development to PAC with

32 MPa but the latter's density (2120 kg/m^3) is 41.3% higher than CNF-LCC (1500 kg/m^3). When compared with PAC of lower densities, CNF-LCC shows much lower overall specific creep and the 270-day specific creep of CNF-LCC ($61.7 \text{ } \mu\epsilon/\text{MPa}$) is 52.3%, 68.7% and 82.9% lower than PAC with 1880 kg/m^3 , 1650 kg/m^3 and 1410 kg/m^3 density, respectively. In general, CNF-LCC has comparable or even better creep restraint capacity than NWC and LWC at the same compressive level. This is because of the optimal mix design in CNF-LCC. The low water-cement ratio, optimal packing density of solid materials and introducing CNFs effectively reduce capillary porosity in the UHPC base mix, which reduces the probability of gel water movement. Meanwhile, low water-cement ratio and adding CNFs can increase the amount of high density C-S-H which has low creep (Barbhuiya and Chow, 2017, Hewlett and Liska, 2019). In addition, low water-cement ratio resulted in the presence of unhydrated cement phase which can provide additional restraint (Lopez, 2005). The curing process and loading environment of CNF-LCC may also contribute to lower specific creep. Unlike the moisture or water curing process of NWC and LWAC as summarised in Table 5.2, the air-curing process of CNF-LCC reduced the moisture content in specimens at the time of loading, which means a lower creep potential (Pihlajavaara, 1974, Neville and Brooks, 1987). At the same time, the same curing and testing condition make CNF-LCC samples reach hydal equilibrium with the surrounding environment before the load application and it can reduce the drying creep (Neville, 1959). Furthermore, it is found that the humidity of testing condition

for CNF-LCC (70% RH) was higher than those summarised in Table 5.2 and this may decrease the creep behaviour (Wendling et al., 2018, BSI, 2004).

Table 5.2 Summary of creep tests on NWC and LWC from literature

Concrete type	Authors (year)	Solid material composition	w/c	ρ_d (kg/m ³)	Curing condition	Sample size	σ_c applied age (days)	f_{cm} (MPa)	E_c (GPa)	σ_c/f_{cm}	T / RH at loading
NWC	Best and Polivka (1959)	Cement, natural sand-gravel aggregates	0.41	2419	Fog room at 21 °C	Ø150×900 mm cylinder	42	34.5	24.8	40%	21 °C / 50%
	Van der Wegen and Bijen (1985)	Cement, river sands, river aggregates	0.53	2318	7-day water curing and 21-day curing at 21 °C and 50% RH condition	100×100×400mm prism	28	33.6	27.7	30%	20 °C / 50%
	Tang et al. (2014)	Cement, normal coarse aggregates and sands	0.5	2325	Water curing at 27 °C	Ø150×300 mm cylinder	28	55	34.2	30%	25 °C / 50%
	Wendling et al. (2018)	Cement, sands, limestone	0.37	2371	23 °C and 50% RH	Ø200×1245 mm cylinder	28	53	-	40%	20 °C / 10% ~ 75%
LWAC	Best and Polivka (1959)	Cement, natural sand, expanded-shale A	0.41	1794	Fog room at 21 °C	Ø150×900 mm cylinder	42	34.5	17.9	40%	21 °C / 50%
		Cement, natural sand, expanded-shale B		1746				34.5	15.2		
		Cement, natural sand, expanded-shale C		1778				34.5	15.2		
	Van der Wegen and Bijen (1985)	Cement, river sands, Lytag aggregates Cement, river sands, Aardelite aggregates	0.53	1961 2071	7-day water curing and 21-day curing at	100×100×400mm prism	28	37.6 30.1	22.7 18.9	30%	20 °C / 50%

21 °C and 50% RH condition											
Polystyrene aggregate concrete (PAC)	Lopez et al. (2004)	Cement, fly ash, silica fume, sands, expanded slate	0.23	1875	Accelerated curing	Ø100×380 mm cylinder	1	67.3	28.5	40%	21 °C / 50%
	Wendling et al. (2018)	Cement, sands, expanded shales	0.36	1999	23 °C and 50% RH	Ø200×1245 mm cylinder	28	46	/	40%	20 °C / 10% ~ 75%
	Tang et al. (2014)	Cement, sands, normal coarse aggregates, PA	0.5	2120	Water curing at 27 °C	Ø150×300 mm cylinder	28	32	24.1	30%	25 °C / 50%
		Cement, sands, normal coarse aggregates, PA	0.5	1880				21	18.1		
		Cement, sands, normal coarse aggregates, PA	0.5	1650				15.2	14.5		
		Cement, sands, normal coarse aggregates, PA	0.5	1410				9.3	9.1

Figure 5.6 Comparison between CNF-LCC and NWC from literature

Figure 5.7 Comparison between CNF-LCC and LWAC from literature

Figure 5.8 Comparison between CNF-LCC and PAC from literature

5.3.3.3 Prediction of creep and model comparison

Creep-time relationship of concrete has been predicted by many analytical models including ACI 209R-92 (2008), GL2000 (2004), Eurocode 2 (2004) and CEB-FIB Model Code (2010). All of these models predict creep strain in the form of creep coefficient which is a dimensionless quantity and is equal to the ratio of creep strain to initial elastic strain. The experimental values of creep coefficient of CNF-LCC are compared with the predicted values from these four models and the results are shown in Figure 5.9. As indicated in the figure, ACI 209R-92 and GL2000 models greatly overestimated the creep coefficient of CNF-LCC at all ages, especially after the initial days under sustained loads. The great deviation of GL2000 model was expected because this model is meant to predict the creep of NWC but not lightweight concrete.

Even though ACI 209R-92 model can be applied to NWC and LWAC, it still gives greatly overestimated results because ACI model is very dependent on the mixture compositions of concrete but CNF-LCC has totally different mix design system from NWC and LWAC. CEB-FIB Model Code shows the best overall prediction closely followed by EC 2 model. Although CEB-FIB Model Code overestimates the creep in the initial period less than 90 days and underestimates the creep for the period greater than 90 days, it gives the best prediction performance with the experimental results of CNF-LCC. CEB-FIB Model Code model does not require the information about mixture compositions except the cement type and it mainly depends on the age at loading, compressive strength, relative humidity during tests and sample size. EC 2 model shows the second best estimation and it follows the same tendency as CEB-FIB Model Code model. EC2 model has similar equations to CEB-FIB Model Code model but it does not have the adjustment for the temperature effect because a higher temperature will result in higher creep. This is why CEB-FIB Model Code has more accurate results in the prediction of long-term creep than EC 2 model. The predicted one-year creep coefficient of CNF-LCC by these models is listed in Table 5.3. The one-year creep coefficient predicted by CEB-FIB Model Code model is found the closest to the experimental data because the ratio of it to the experimental data is 0.9. In order to predict the creep of CNF-LCC more accurately, a hyperbolic expression was employed for prediction as follows:

$$c(t) = \frac{t}{A+Bt} \quad (5.2)$$

where $c(t)$ is the creep coefficient and t is the loaded age; A and B are constants which can be determined from short-term experimental data. A plot of $t/c(t)$ against t gives a straight line in which B is the slope and A is the intercept of the $t/c(t)$ axis. When t tends to infinity, the ultimate creep coefficient is given by $1/B$. Based on the experimental results, the values of A and B can be obtained by the trendline as shown in Table 5.3 with the R^2 value. As Figure 5.9 indicates, Equation 5.2 has the best agreement with the experimental data compared with other analytical models. Therefore, the ultimate creep coefficient of CNF-LCC can be calculated by Equation 5.2 and the value is 1.22 as shown in Table 5.3. According to Table 5.3, CEB-FIB Model Code model gives the most accurate prediction of the ultimate creep coefficient which has a ratio as high as 0.97 to the experimental data (Equation 5.2).

Figure 5.9 Comparison between experimental and analytical results

Table 5.3 Estimated creep for CNF-LCC by different models

Parameter	Measured	Equation 5.2			ACI 209R-92	GL2000	EC 2	CEB MC 2010
		A	B	R ²				
		47.57	0.82	0.99				
One-year creep coefficient	1.05		1.05		1.52 (1.45*)	1.49 (1.42*)	0.90 (0.86*)	0.94 (0.90*)
Ultimate creep coefficient			1.22		1.71 (1.40#)	2.05 (1.68#)	1.11 (0.91#)	1.18 (0.97#)

Note: the number marked by “*” is the ratio of predicted one-year creep coefficient to the measured one; the number marked by “#” is the ratio of predicted ultimate creep coefficient to one calculated by Equation 5.2.

5.4 Summary

The long-term properties of CNF-LCC including durability, shrinkage and creep are measured and discussed in this chapter. The experimental results indicated that CNF-LCC can work as a type of long-lasting construction material and CNFs showed significant improvement in these properties. The conclusion of CHAPTER 5 are as follows:

1. Low capillary porosity in the UHPC & ceUHPC base mix and unconnected foam bubbles caused water penetration depth and water permeability coefficient of LCC & CNF-LCC to be much lower than NWC. Besides, CNFs can significantly reduce water penetration and water permeability coefficient of LCC by 45.8% and 70.6%, respectively. The main reason was that capillary pores that control the permeability of concrete were reduced due to the filler function of CNFs.

2. Both CNF-LCC & LCC showed higher autogenous shrinkage than NWC & LWAC due to a higher amount of unhydrated cement in the ceUHPC/UHPC base mix. However, the drying and total shrinkage of CNF-LCC & LCC were lower than NWC & LWAC because of low water/cement ratio and high packing density of the base mix, as well as shrinkage-reducing nature of the foam agent. Based on the capillary tension theory, CNFs can further reduce the finer capillary porosity due to their nanoscale dimension, which was evidenced by the MIP test results.

3. CNF-LCC showed comparable or even better creep resistance than NWC and LWAC with similar compressive strength. However, higher creep in CNF-LCC was observed when compared with higher strength NWC and LWAC. As a result of the absence of creep data for conventional foam concrete, CNF-LCC was compared with similar lightweight concrete and exhibited better performance even when the strength or density is similar. Moreover, CNFs can effectively reduce creep because they decreased the pathway for the movement of gel water to capillary pores and increased

the amount of high-density C-S-H at the cost of low-density C-S-H. Although CEB-FIB Model Code model provided relatively accurate predictions compared with those from other codes, a much more precise hyperbolic expression model was proposed to describe the creep development of CNF-LCC and the ultimate creep coefficient was predicted.

So far, the mix design, short-term and long-term engineering properties of CNF-LCC have been reported from CHAPTER 3 to CHAPTER 5. Prior to investigating the structural performance of reinforced CNF-LCC members, it is essential to study the bond behaviour between CNF-LCC and steel reinforcement. The bond behaviour will not only ensure the composite action of concrete and reinforcement but also influence the structural performance at both serviceability and ultimate limit state. The investigated contents about the bond behaviour of reinforcement in CNF-LCC are showed in the next chapter.

CHAPTER 6 BOND BEHAVIOUR BETWEEN CNF-LCC AND DEFORMED STEEL REINFORCEMENT

6.1 Introduction

Bond stress between concrete and reinforcing bars significantly influences the performance of reinforced concrete (RC) structures. At serviceability limit state, bond stress affects the location, spacing, width and length of cracks in concrete (Muhamad et al., 2012, CEB-FIP, 2010). In addition, tension stiffening of cracked concrete is induced by bond and further influences the curvature and deflection of RC members (Muhamad et al., 2012, Park and Paulay, 1975, CEB-FIP, 2010). At ultimate limit state, bond strength directly determines the embedment and lap lengths of reinforcement. Moreover, bond strength is responsible for load-carrying capacity while slip of reinforcement contributes to rotation capacity at beam-column joints (Sezen and Setzler, 2008, CEB-FIP, 2010). As CNF-LCC is a different material from NWC and LWAC, it is necessary to study the bond resistance of steel bars in CNF-LCC prior to investigating the structural performance of reinforced CNF-LCC members.

In this chapter, the bond behaviour between well-confined CNF-LCC and deformed steel bars is studied by the pullout test which is widely used in the laboratory due to its ease of fabrication and simplicity. There were two series of pullout tests to study the bond behaviour between CNF-LCC/LCC and steel reinforcement at the elastic

and post-yielding stage respectively. In the first series of pullout tests, bond behaviour between well-confined CNF-LCC/LCC and deformed steel bars at the elastic stage was studied. The primary objective was to obtain local bond stress-slip relationships at the elastic stage of the bar and the test results were compared with NWC, LWAC and traditional foam concrete of similar compressive strength from literature. In addition, the effect of bar diameter and CNFs on the elastic bond behaviour was investigated in this series of tests. In the second series of pullout tests, bond behaviour between well-confined CNF-LCC and post-yield deformed steel bars was studied because the bond stress was significantly reduced due to the changing geometry of steel bars after yielding. The primary objectives were to obtain the local bond stress-slip relationship at the post-yield stage of the bars and the development length as well as force-slip relationship of steel bars embedded in CNF-LCC. In order to predict these experimental results, a new model was derived from the control field equation of bond behaviour and was characterised by low computational effort and high accuracy. Meanwhile, the proposed model had a wide range of applications for both adequate and inadequate embedment length conditions. The predictions of the proposed new model were validated with experimental results from the present study and other publications.

6.2 Test program

6.2.1 Design of test specimens

The details of the pullout specimens are listed in Table 6.1 and shown in Figure 6.1 to Figure 6.2. In the first series of pullout tests, short embedment length was used to study the bond behaviour at the elastic stage of steel bars. Four different bar diameters including 13, 16, 20 and 25 mm were prepared to study the influence of bar diameter on elastic bond behaviour. In addition, the effect of CNFs was investigated by two groups of pullout test samples embedded by $\Phi 16$ mm bars but cast by CNF-LCC and LCC, respectively. To study the bond behaviour between CNF-LCC and steel bars at post-yield stage, $\Phi 13$ mm steel bars with long embedment length were used in the second series of tests. All the specimens were designed to simulate the condition found in beam-column joints where confinement is guaranteed to the pullout tests (Eligehausen et al. (1982)). For each of the specimens, two or three identical specimens were tested and the average results were reported.

Single deformed steel bars were embedded in the middle position of the CNF-LCC/LCC blocks and cast in horizontal position. In the first series, a short length of steel bar was embedded in CNF-LCC/LCC blocks to ensure elastic stage and pullout failure of the bars. This embedment length was selected as five times the bar diameter because this value is short enough so that the above requirement and assumption are satisfied but also long enough to reduce the scatter of test results which are commonly

observed in tests with very short embedment lengths (Eligehausen et al., 1982). The distribution of bond stress and bar slip can be assumed uniform along the short embedment length of five times the bar diameter and they can be easily calculated from the initially measured results. In the second series, the embedment length should be much longer than $5d_b$ to allow inelastic development and rupture failure of steel bars. Therefore, the embedment length of steel bars was increased to 25 times the bar diameter. However, long embedment length nullifies the assumption of uniform stress distribution in the first test series and it was difficult to directly measure the local bond stress and slip along the long embedment length. The solution was to measure the bar strain distribution along the embedment length by strain gauges and convert it to the distribution of bond stress and slip. The embedded steel bars were grooved on both opposite sides along the longitudinal ribs with 4.5 mm width and 2.5 mm depth as shown in Figure 6.2 (b). Post-yield strain gauges with 2 mm gauge length were mounted along the grooves at an interval of 2.5 times of bar diameter as shown in Figure 6.2 (b). Afterwards, epoxy was applied to fill in the grooves which can protect the strain gauges from being damaged during casting. Under different load steps, the reading of strain gauges gave the bar strain distribution along the embedment length. It can be used to deduce the distribution of bar stress and bar slip by the stress-strain relationship of the bar and the integral relationship. The bond stress in each bar segment between two adjacent mounted strain gauges can be

assumed constant and calculated by the force equilibrium of this segment. In this way, the distribution of bond stress can be obtained.

Sufficient confinement was provided via a thick concrete cover (250 mm) and secondary reinforcement (vertical bars and links) to limit splitting cracks. Four deformed vertical bars of 10 mm diameter were rigidly connected with the 10 mm thick top and bottom steel plates by welding to ensure good anchorage. Links of 6 mm diameter were positioned at 100 mm centre-to-centre spacing. Under pull-out loading, local bond failure was caused by the separation of concrete cones at the active end of embedded bars due to bond force acting on concrete keys in between the ribs (Engström et al., 1998, Viwathanatepa et al., 1979). The depth of the concrete cone was approximately five times the diameter of steel bar, which reduced the embedment length (Soltani and Maekawa, 2008). To avoid it, PVC pipes with a typical length of five times the bar diameter were provided at each end of embedded steel bars for both series of test specimens. The inner diameters of the PVC pipes were a little larger than the nominal bar diameter (including ribs) of the embedded bars. The PVC pipes neither restrained the slip of the bar nor markedly influenced the bond transfer from the bar to the concrete.

Table 6.1 Details of the first series pullout tests specimens

Series	Specimen	Number	Concrete	Bar diameter d_b (mm)	Embedment length l (mm)
First	SA-13	3	CNF-LCC	13	$5d = 65$
	SA-16	2	CNF-LCC	16	$5d = 80$
	SB-16	2	LCC	16	$5d = 80$
	SA-20	3	CNF-LCC	20	$5d = 100$
	SA-25	3	CNF-LCC	25	$5d = 125$
Second	LA-13	2	CNF-LCC	13	$25d = 325$

Definition of the specimen label: “S” – Short embedment length; “L” – Long embedment length; “A” – Sample cast by CNF-LCC; “B” – Sample cast by LCC; “number” – Bar diameter.

(a)

(b)

Figure 6.1 Schematic diagrams of pullout test specimens with short embedment length (a) $\Phi 13$ mm bar; (b) $\Phi 16$ mm bar; (c) $\Phi 20$ mm bar; (d) $\Phi 25$ mm bar (all units in mm)

Figure 6.2 Schematic diagrams of pullout test specimens with long embedment length (a) Details of specimens; (b) details of $\Phi 13$ mm steel bar and layout of strain gauges along the bar

6.2.2 Test set-up and instrumentation

To fix the CNF-LCC/LCC blocks and apply a tensile pull-out force on the embedded steel bar, a test set-up was designed as shown in Figure 6.3. Four PVC pipes with 20 mm diameter were embedded at four corners of the concrete block, through which four steel bolts were placed to connect the top and the bottom steel plates together. A circular opening with 80 mm diameter was cut on the top steel plate to reduce compressive force (which is beneficial for bond strength). A similar circular opening (60 mm diameter) was also made on the bottom plate to install measurements for the slip at the free end of the embedded bar. The testing machine with a capacity of 200 tons in NTU Construction Laboratory is shown in Figure 6.4. During the tests, the bottom jig of the testing machine clamped the bottom steel plate to fix the CNF-LCC/LCC block, while the top jig gripped the embedded bar to apply a pull-out force. A displacement-control loading was adopted at 2 mm/min which is a standard pull-out rate.

Figure 6.3 Set-up for pullout tests (all units in mm): (a) front view of set-up; (b) side view of set-up; (c) top steel plate; (d) bottom steel plate

Figure 6.4 Testing machine for pullout tests (all units are in mm)

Applied loading and axial displacements were directly recorded by the testing machine. In addition, five linear variable differential transducers (LVDTs) were installed at different positions. LVDT A (see Figure 6.3) measured the slips at the free end of the embedded bar from the circular opening on the bottom steel plate. To measure the bar slips at the loaded end, an aluminum plate was fixed onto the loaded bar as shown in Figure 6.4 and LVDT B and C were used to measure the displacement of the aluminum plate which was nearly equal to the loaded end bar slip. Meanwhile, two other LVDTs (D and E) were positioned at the bottom steel plate to show that the CNF-LCC/LCC block did not tilt or move vertically.

6.2.3 Material properties

Three $\Phi 150 \times 300$ mm cylinders were cast from each batch to determine the properties of the concrete mixture used and the average results along with respective standard deviation are presented in Table 6.2. The tensile stress-strain relationships of the steel bars with different diameters (13, 16, 20 and 25 mm) were also measured. During these tension tests, both extensometer and steel strain gauges were used to measure average strain in the gauge length. Three identical rebars were tested for each bar diameter and the average values were listed in Table 6.3.

Table 6.2 Average density and compressive strength of CNF-LCC from each batch

Series	Batch	Concrete	Average density (kg/m ³)	Average Compressive strength (MPa)
First	SA-13	CNF-LCC	1540 (9.32*)	31.2 (0.52*)
	SA-16	CNF-LCC	1530 (8.21*)	30.6 (0.47*)
	SB-16	LCC	1540 (9.07*)	26.5 (0.53*)
	SA-20	CNF-LCC	1520 (6.51*)	32.2 (0.36*)
	SA-25	CNF-LCC	1520 (7.66*)	32.0 (0.43*)
Second	LA-13	CNF-LCC	1540 (6.41*)	31.2 (0.52*)

Noting: * represents the standard deviation

Table 6.3 Material properties of the steel bars

Material properties	Nominal bar size (mm)	Elastic modulus (MPa)	Yield strength (MPa)	Yield strain (%)	Hardening Modulus (MPa)	Ultimate strength (MPa)	Ultimate strain (%)
H13	13	190572	564.9	0.297	2087	672.7	5.468
H16	16	196931	570.5	0.290	1271	674.7	8.511
H20	20	187453	618.7	0.330	1102	717.9	9.340
H25	25	179971	620.9	0.345	786	776.1	20.1

6.3 Test results and discussion

6.3.1 First series of pullout tests (at the elastic stage of bars)

The initial test results were the load-slip relationship and local bond stress-slip curves can be deduced from them. The measured free and loaded end bar slips did not differ significantly from each other due to the short embedment length. Hence, the loaded end slips were chosen to represent local slips in the present study. The applied force can be converted into local bond stress using Equation 6.1:

$$\tau = F/(\pi d_b l) \quad (6.1)$$

where F is the applied force, d_b is the bar diameter and l is the embedment length of the steel bars ($5d_b$). All the specimens were failed by pullout and there were no splitting cracks observed on the surface. The test results were presented according to the two parameters, viz. CNFs and bar diameter.

6.3.1.1 Effect of CNFs

The experimental τ -s curves of CNF-LCC and LCC samples are shown in Figure 6.5. Besides, a typical τ -s relationship of NWC is also plotted according to commonly accepted Eligehausen's model (CEB-FIP, 2010) for comparison purpose. In addition, the experimental bond strength of CNF-LCC samples and LCC samples along with the bond strength of NWC and LWAC from standards are summarised in Table 6.4.

Figure 6.5 Local bond stress-slip relationship of Φ CNF-LCC and LCC with 16 mm steel bars

Table 6.4 Bond strength (τ_m) of CNF-LCC, LCC, NWC, and LWAC

Specimen	Bond strength (τ_m) (MPa)		
CNF-LCC-1	13.7	Average	13.2
CNF-LCC-2	12.6		
LCC-1	10.4	Average	11.0
LCC-2	11.5		
NWC ($f_{cm} = 30$ MPa)*	13.7		
LWAC ($f_{cm} = 30$ MPa)*	8.8~13.7		

Noting: The bond strength of NWC is equal to $2.5\sqrt{f_{cm}}$ (CEB-FIP, 2010). The bond strength of LWAC ranges from nearly equal to 65% of that obtained from NWC (ACI Committee 213, 2003).

It can be observed from Figure 6.5 that the shapes of experimental τ -s curves of CNF-LCC and LCC samples were almost the same as the typical τ -s relationship of NWC. The initial τ -s relationship was nearly linear and it gradually deviated from linearity and became horizontal when approaching the bond strength. After the peak value, the curve showed gradual descending behaviour, and finally maintained a stable residual (friction) bond strength. The reasonable curve shape indicated proper sample design and testing procedure for determining the bond strength of reinforcing bars.

Eligehausen et al. (1982) reported that the bond resistance mainly depends on the mechanical interlocking between the bar ribs and the concrete. With increasing applied load, higher bond resistance is provided by crushing of local concrete keys in between the ribs when the bar slips relative to surrounding concrete. Hence, the bearing capacity of the concrete keys will determine the bond resistance between the

concrete and the steel bars. However, the radial component of the mechanical locking force induces tensile hoop stresses which result in splitting cracks in surrounding concrete, which will significantly reduce the bond stress. The study of the bond behaviour of cellular concrete is very scarce in the literature. Lim (2007) investigated the bond strength between the foam concrete (1350 kg/m³ density and 20 MPa cube strength) and Φ 16 mm deformed steel bar (embedment length from 50 to 130 mm). He concluded that foam concrete shows much lower bond strength (less than 2 MPa) than NWC and LWAC due to introduced air bubbles reducing the effective contact area at the concrete-rebar interface, and thus larger anchorage length of rebar in foam concrete is needed. A similar conclusion was obtained by de Villiers et al. (2017) who conducted pull-out tests on foam concrete and some specimens showed splitting failure. However, the bond strengths of both CNF-LCC and LCC exceed those from the literature and were comparable with LWAC as listed in Table 6.4. Moreover, the average bond strength of CNF-LCC was only 3.6% lower than NWC, especially specimen CNF-LCC-1 which showed similar bond-slip behaviour and equivalent bond strength with NWC. This is because the compressive and flexural strength of UHPC based LCC and CNF-LCC are higher than traditional foam concrete and are comparable with NWC and LWAC. The denser structure of the pore walls can also increase the effective contact area at the foam concrete-rebar interface. In addition, the use of lower water/cement ratio in the base mix could also enhance bond performance by improving adhesion, reducing shrinkage and minimising bleeding

around the steel bar (Bogas et al., 2014). When comparing the results of CNF-LCC and LCC specimens, the whole range of the bond stress was improved and the bond strength (maximum bond stress) was increased by 20% with CNFs. Based on the above-mentioned bond mechanism, adding CNFs increases the compressive strength and bearing capacity, which directly increased the bond strength with reinforcement. Besides, the increased flexural tensile toughness and tensile strength of CNF-LCC delay the rate of crack opening during bond-slip (de Villiers et al., 2017) and improve the resistance to splitting cracks respectively, which indirectly increase the bond resistance.

6.3.1.2 Effect of bar diameter

The average experimental local bond stress-slip relationships for $\Phi 13$, $\Phi 16$, $\Phi 20$ and $\Phi 25$ mm steel bars are shown in Figure 6.6. The local bond stress-slip curve shapes of the specimens with different bar diameters were almost the same as those from Eligehausen's tests for NWC as introduced in Section 6.3.1.1. The bond strength and corresponding bar slip obtained from Figure 6.6 are summarised in Table 6.5. The bond strength of LWAC ($f_{cm} = 30$ MPa) is included in Table 6.5 which extends from 65% to 100% of that obtained with NWC (ACI Committee 408, 2003).

Figure 6.6 Local bond stress-slip relationship of CNF-LCC with $\Phi 13$, $\Phi 16$, $\Phi 20$ and $\Phi 25$ mm steel bars

Table 6.5 The bond strength and corresponding bar slip of specimens

Specimen	Bar diameter (mm)	Average bond strength (MPa)	Average bar slip at bond strength (mm)
SA-H13	13	13.9	1.4
SA-H16	16	13.2	2.6
SA-H20	20	9.6	1.8
SA-H25	25	9.7	2.1
NWC ($f_{cm} = 30$ MPa)*	-	13.7	1.0
LWAC ($f_{cm} = 30$ MPa)*	-	8.8~13.7	-

It was found that the specimens with $\Phi 13$ mm steel bars showed the highest bond strength of 13.9 MPa followed by the specimens with $\Phi 16$ mm steel bars which had slightly lower bond strength (13.2 MPa). When the bar diameter increased to 20 mm, the bond strength was significantly reduced by 30.9% and 27.3% compared with $\Phi 13$

and $\Phi 16$ mm steel bars respectively. Further increasing the bar diameter to 25 mm, however, there was almost no changing of bond strength. It can be summarised that the bond stress between CNF-LCC and steel bars was reduced with increasing bar diameter. This trend is similar to the influence of bar diameter on bond characteristics between NWC and deformed steel bar interface, which has been studied by many researchers (ACI Committee 408, 2003). The bonding area and tension force of steel reinforcement are proportional to the perimeter and area of the reinforcement cross section, respectively. The relative bonding area of steel reinforcement could be represented by the ratio between the perimeter and area of the cross section, which is equal to $4/d_b$ (Hao and Jian, 2012). Therefore, increasing bar diameter (d_b) would reduce relative bonding area of steel reinforcement and the bond strength is decreased consequently. In addition, increasing bar diameter would result in more serious bleeding around the surface of steel bar and thus increase the thickness of transition zone between the reinforcement and surrounding concrete (Barbosa et al., 2008). The thicker and more porous transition zone could decrease the bearing capacity of the concrete keys between the ribs and reduce the bond strength consequently. However, the reduction of bond strength in NWC is slight or even negligible with increasing bar diameter. In the pullout tests by Eligehausen et al. (1982), the maximum bond resistance was reduced by 6% and 15% when the bar diameter was increased from 19 mm to 25 mm, and 19 mm to 32 mm, respectively. Martin (1973) reported that the influence of bar diameter was slight based on a large number of pullout tests.

Meanwhile, EC2 also indicates that increasing bar diameter will not change the bond strength when the bar diameter is not larger than 32 mm (BSI, 2004). Based on the experimental results in Table 6.5, it should be more careful to use steel bars with diameter greater than or equal to 20 mm in CNF-LCC because of the obvious reduction of bond strength. Longer development length and splice length will be required for steel bars with diameter greater than or equal to 20 mm embedded in CNF-LCC. From the perspective of bond strength, it is recommended to use more reinforcing bars of smaller diameter rather than fewer bars of larger diameter in reinforced CNF-LCC members. The bar slip at the bond strength of NWC (1 mm) is always lower than those of CNF-LCC with different bar diameters. Moreover, the stiffness of ascending branch of NWC is higher than all the CNF-LCC samples as shown in Figure 6.7. These are attributed to lower elastic modulus of CNF-LCC compared with that of NWC.

Figure 6.7 Ascending branches of the local bond-slip curves of CNF-LCC and NWC

6.3.2 Second series of pullout tests (at the post-yield stage of bars)

6.3.2.1 Force slip relationship

Unlike the pullout failure in the first series of pullout tests (short embedment length), the embedded reinforcement was fractured in the second series of tests because of the long embedment length. At the same time, significantly different force and slip relationships were obtained in this series of tests as shown in Figure 6.8. Lee et al. (2016) utilised the similar design of pullout samples and test set-up to conduct pullout tests of $\Phi 13$ mm steel reinforcement with long embedment length in NWC. The 28-day cylinder compressive strength of NWC in Lee et al. (2016) was 31.8 MPa which is close to that of CNF-LCC. However, the embedment length used in Lee et al. (2016) was 20 times bar diameter. The force-slip relationship from Lee et al. (2016) was

compared with CNF-LCC as plotted in Figure 6.8. It is found that the applied force and loaded end slip relationship of CNF-LCC and NWC (Figure 6.8 (a)) was similar and the embedded reinforcement in both of them experienced yielding followed by softening and eventual fracture at the onset of grooved sections. In addition, the loaded end slip of long reinforcement at corresponding critical loads was summarised in Table 6.6. Compared to NWC, the critical force in CNF-LCC samples was a little lower because of the slightly higher mechanical properties of $\Phi 13$ mm steel bars used in Lee et al. (2016). The corresponding slips at yielding and ultimate force in CNF-LCC and NWC samples were very close. However, CNF-LCC samples showed slightly higher slip at fracture point than NWC. This is attributed to the lower bond strength at post-yielding stage in CNF-LCC, which was presented in the later section. The free end slip of long reinforcement was also recorded by LVDT A in Figure 6.3. The relationship between applied force and free-end slip of long reinforcement was exhibited in Figure 6.8 (b). It was found that the free end slip of long reinforcement in both CNF-LCC and NWC increased with applied force until the ultimate force. After that, however, the free end slip decreased due to necking of reinforcement up to fracture of the steel bar. The maximum free end slips of reinforcement in CNF-LCC and NWC were very close and they were 0.46 mm and 0.51 mm, respectively. Experimental results indicated that the embedment length of reinforcement in this study (25 times the bar diameter) was insufficient to ensure zero slip at the free end.

(a)

(b)

Figure 6.8 Force and slip relationship of long embedded reinforcement (a) loaded end slip; (b) free end slip

Table 6.6 Loaded end slip at critical loads

Specimen	Yield force F_y (kN)	Slip at F_y (mm)	Ultimate force F_u (kN)	Slip at F_u (mm)	Fracture force F_r (kN)	Slip at F_r (mm)
LA-13 (CNF-LCC)	61.8	1.15	73.8	4.53	50.2	10.02
NWC (Lee et al., 2016)	56.4	1.09	67.9	4.40	44.5	10.81

6.3.2.2 Steel strain profiles

In addition to applied load versus slip relationship of embedded long reinforcement, the steel strain distribution at each loading stage was also measured by the strain gauges as shown in Figure 6.2 (b). Figure 6.9 (a) shows the steel strain profile at the elastic stage of embedded reinforcement. It was found that only limited embedment length of steel reinforcement was mobilised to transfer the bond stress to surrounding concrete when the applied load was half of the yield strength of reinforcement. With increased applied force, longer embedment length of reinforcement was mobilised and the bond stress was propagated to the free end. At the yield point as shown in Figure 6.9 (a), almost the full embedment length of reinforcement attained the tensile strain except for the free end in which the strain was still zero. Thereafter, the strain close to the loaded end increased significantly when the applied load entered the post-yield stage of embedded reinforcement as shown in Figure 6.9 (b). When the ultimate load of embedded reinforcement was reached, the length of post-yield steel reinforcement was about 92.5 mm and the remaining length was still at the elastic stage.

(a)

(b)

Figure 6.9 Steel strain profile along the embedded long reinforcement in CNF-LCC (a) at the elastic stage; (b) at the post-yield stage

6.3.2.3 Bond stress profiles

With the obtained steel strain profile in Section 6.3.2.2, the steel stress distribution can be obtained by the assumed bilinear stress-strain relationship of steel reinforcement: (1) elastic state $\sigma_s = E\varepsilon_s \leq \sigma_y$ and (2) post-yield state $\sigma_s = \sigma_y + E_h \cdot (\varepsilon_s - \varepsilon_y) \geq \sigma_y$. The terms ε_s and ε_y are respectively the tensile strain and the yield strain of reinforcement; E and E_h are respectively the elastic modulus and hardening modulus of reinforcement, and σ_y is the yield strength of reinforcement. The bond stress distribution between two steel strain gauges can be assumed constant due to the short distance and it can be calculated according to the force equilibrium in each steel segment. Figure 6.10 (a) and (b) present the bond stress profile along the embedded reinforcement at the elastic and post-yield stage. When the applied force reached half of the yield force of the reinforcement, only part of the embedded reinforcement was mobilised to transfer steel stress to surrounding CNF-LCC and the bond stress was nearly zero from the free end to a length of 92.5 mm as shown in Figure 6.10 (a). With increased applied force, longer embedded reinforcement and higher bond stress were mobilised to transfer applied stress. At the yielding point of reinforcement, the maximum bond stress at the loaded end was about 13.0 MPa which is very close to the bond strength (13.9 MPa) obtained in the pullout test with short embedment length. After the bar had yielded, however, the rapidly increasing reinforcement strain and the reduced reinforcement cross-section due to Poisson's

Ratio significantly reduced the bond stress and the bond stress at the loaded end was only 4.1 MPa as shown in Figure 6.10 (b). This phenomenon conformed to the experimental results of NWC from Eligehausen et al. (1982) and Viwathanatepa et al. (1979). They also suggested that the bond stress along the post-yield reinforcement in NWC can be assumed as constant with acceptable error, which was also suitable for CNF-LCC based on the results in Figure 6.10 (b). Therefore, an average value of 3.5 MPa was taken to represent the bond stress at the post-yield stage in CNF-LCC. This value was lower than that of NWC which is equal to 70-80% of friction bond stress. Furthermore, the maximum bond strength of 13.5 MPa was observed in the elastic length of reinforcement and this value was almost equal to the bond strength in the first series of pullout tests.

(a)

Figure 6.10 Bond stress profile along the embedded long reinforcement in CNF-LCC (a) at the elastic stage; (b) at the post-yield stage

6.3.3 Analytical model for bond-slip behaviour

6.3.3.1 Previous analytical models

There are generally two proposed analytical models to solve bond stress problems. The main differences among them are the definition of bond stress distribution and solution procedure.

In macro models, the bond stress distribution is assumed uniform or stepped along the embedment length of the rebar. Due to this assumption, the bar stress distribution along the rebar is linear as shown in Figure 6.11. It significantly simplifies the computation procedure. Representative models of this kind are proposed by Sezen *et*

al. (2008) and Alsiwat *et al.* (1992). However, actual bond stress distribution varies pointwise and the uniform stress profile only appears at the maximum applied tensile force (Kang and Tan, 2015). The uniform bond stress assumption may cause errors in analytical predictions. In this condition, Sezen *et al.* (2008) indicated that the accuracy of macro models is significantly reduced if the free-end slip is high and they proposed a minimum embedment length for using macro models. Therefore, macro models can only be applied for adequate embedment length condition.

Figure 6.11 Calculation diagram of macro models

Micro models analyse the steel-concrete interfacial stress at the local level, and the essential difference between macro and micro models is a varying τ - s relationship with a numerical model (Sezen and Setzler, 2008). After determining the τ - s relation at both elastic and post-yield states of rebars, a nested iteration loop is applied to derive bond-related problems. In this procedure, the embedded rebars are divided into small segments with equal length as shown in Figure 6.12. The term Δl is the length of the steel segment; σ_e and σ_h are stresses at both ends of the steel segment; ε_e and ε_h are the strains at both ends of the steel segment; s_e and s_h are the

respective slips at both ends of the steel segment. The first and most important step of this iteration procedure is to assume the values of bar slips at the ends of each segment. Due to the finite length of each segment, bond stress can be assumed constant while bar strain varies linearly. The bond stress along each segment can be obtained from the assumed slip at the middle point of the segment according to the τ - s relationship. The assumed bar slips are determined when force equilibrium and compatibility between bar slip and strain of the segment are satisfied. The same procedure will be employed for adjoining segments. Although micro models agree well with experimental results compared to macro models, the nested iteration procedure makes them computationally inefficient.

Figure 6.12 Equilibrium (a) and compatibility (b) of small steel segment

6.3.3.2 Analytical model by control field equation

The control field equation builds the relationship between bond stress and bar stress in a steel segment with infinitesimal length dx extracted from an embedded bar as shown in Figure 6.13. It can be deduced according to the force equilibrium of the steel segment given by Equation 6.2. A_s and P_s are cross-sectional area and perimeter of reinforcement respectively, and σ_s is the tensile stress of reinforcement. The control field equation describes the force transfer from the rebar to surrounding concrete and the bond stress represents the rate of this force transfer.

Figure 6.13 Force equilibrium of an infinitesimal steel segment

$$\frac{d\sigma_s}{dx} - \frac{P_s}{A_s} \cdot \tau = 0 \quad (6.2)$$

σ_s and τ are two unknowns which make Equation 6.2 intractable. The unknowns can become uniform if the stress-strain relationship of steel bars, compatibility between bar strain and slip, as well as τ -s relationship, are substituted into Equation 6.2. The steel bars have a bilinear stress-strain relationship: (1) elastic state $\sigma_s = E\varepsilon_s \leq \sigma_y$ and (2) post-yield state $\sigma_s = \sigma_y + E_h \cdot (\varepsilon_s - \varepsilon_y) \geq \sigma_y$. The terms ε_s and ε_y are respectively the tensile strain and yield strain of reinforcement; E and

E_h are respectively the elastic modulus and hardening modulus of reinforcement, and σ_y is the yield strength of reinforcement. The compatibility between bar strain and slip is shown in Equation 6.3 (Shima et al., 1987):

$$s(x) = s_f + \int_{x_f}^x \varepsilon_s dx \quad (6.3)$$

where s_f is the slip at the free end of reinforcement, x_f and x are the coordinates of the free end and calculated point along the embedded reinforcement, respectively. It is worth noting that the slip is defined as the relative displacement between the rebar and concrete. However, the concrete strain is rather small which can be neglected. Thus, the local slip is approximately equal to bar slip. The author applied Eligehausen's model and uniform stress model to model the bond stress for elastic and post-yield steel bars, respectively. Substituting above-mentioned relationships into the control field equation, the following equations can be obtained:

$$\frac{d^2 s}{dx^2} - \frac{P_s}{EA_s} \cdot \tau(s)_e = 0 \quad \text{when } \sigma_s \leq \sigma_y \quad (6.4)$$

$$\frac{d^2 s}{dx^2} - \frac{P_s}{E_h A_s} \cdot \tau(s)_y = 0 \quad \text{when } \sigma_s > \sigma_y \quad (6.5)$$

where τ_e and τ_y are the bond stress at the elastic and post-yield stages of reinforcement, respectively. Equations 6.4 and 6.5 are second-order differential equations which can be solved once the material properties and boundary conditions are provided. The solutions predict the bar slip distribution $s(x)$ along the elastic

and post-yield segments of the rebar. They were employed to derive the development length and force-slip relationships and the detailed procedure is included in APPENDIX A.

6.3.3.3 Validation of control field equation model

In order to employ the control field equation model, it is necessary to confirm the local bond stress and slip relationship at both elastic and post-yield stage of reinforcement. According to the experimental results in Figure 6.6, Eligehausen model (1982) (Equation A.1 to A.4 in APPENDIX A) was accurate enough to represent the elastic local bond stress and slip relationship in CNF-LCC. The values of corresponding parameters for CNF-LCC and NWC are listed in Table 6.7. The parameters for NWC were from Model Code 2010 (2010) and those for CNF-LCC were based on the results in the first series of pullout tests (SA-13 sample in Figure 6.6). Table 6.7 also gives a constant value to represent the post-yield bond stress τ_y . The constant value for NWC was given by Eligehausen et al. (1982) and Viwathanatepa et al. (1979) while that for CNF-LCC was obtained from experimental results as discussed in Section 6.3.2.3. As discussed in APPENDIX A.3, the force-slip relationship can be divided into four different cases according to the embedment length (Table A.2). The condition in this study fell into Case (2) because of the non-zero free end slip and fracture failure mode, which is similar to the NWC samples from Lee et al. (2016). The control field equation model was verified by predicting

the experimental force-slip relationship of CNF-LCC in Section 6.3.2.1 and NWC in Lee et al. (2016). As for Case (1) and (3) listed in Table A.2, published experimental results of NWC from other researchers were used to validate the predictions of proposed analytical model. The material properties from different tests were listed in Table 6.8. The comparison between experimental and analytical results is shown in Figure 6.14 to Figure 6.16.

The discrepancy between the experimental and analytical results in Figures 6.15 mainly concentrates on the ascending branch although the difference is only a few tenths of a millimeter. This is because the steel reinforcement was grooved for the pullout samples with long embedment length, but the local bond stress-slip relationship used for modelling analysis was obtained from the first series of pullout samples (short embedment length) with original (non-grooved) steel reinforcement. The grooved surface would influence the local bond stress-slip relationship, which resulted in the difference between the experimental and analytical results. In Figures 6.16 (b) and (c), when compared with Ueda's experimental results, the experimental curves of specimens S61 and S107 do not extend as far as the analytical results, as the loads on those specimens were reversed at the maximum displacement (Ueda et al., 1986, Wang et al., 2019). In general, there was a good agreement between experimental and analytical results, which indicated that the control-field equation model is appropriate for adequate and inadequate embedment lengths of

reinforcement for different types of concrete once the local bond stress and slip relationship is determined. In addition, the development length of steel reinforcement embedded in CNF-LCC and NWC can be calculated by the closed-form solution Equation A.20. The development length of $\Phi 13$ mm steel reinforcement in CNF-LCC is about 40 times bar diameter which is slightly greater than that of NWC (36 times bar diameter). It can also be inferred by the experimental results in Section 6.3.2.1 that the CNF-LCC pullout samples with 25 times bar diameter embedment length had a similar bond-slip performance to NWC pullout samples with 20 times diameter embedment length.

Table 6.7 Local bond stress and slip relationship of NWC and CNF-LCC

Material	Maximum bond stress τ_m (MPa)	Slip s_1 (mm)	Slip s_2 (mm)	Frictional bond stress τ_f (MPa)	Slip s_3 (mm)	α	Post-yield bond stress τ_y (MPa)
NWC	$2.5\sqrt{f_{cm}}$	1	2	$0.4\tau_m$	10	0.4	$0.75\tau_f$
CNF-LCC (with $\Phi 13$ mm reinforcement)	13.9	1.3	1.5	3.5	8	0.4	3.3

Table 6.8 Material properties from different tests

Case	Reference	Specimen codes	f_{cm} (MPa)	A_s (mm ²)	P_s (mm)	E (GPa)	σ_y (MPa)	E_h (MPa)	σ_u (MPa)	l (mm)
1 (sufficiently long embedment length)	Bigaj (1995)	P·16·16·1	26.98	174.3	46.8	128.5	539.7	764.4	624.4	Sufficiently long
		P·16·16·2	28.36							
		P·20·16·1	28.36	280.9	59.4	150.3	526.2	799.1	612.9	
		P·20·16·2	26.78							
2 (long embedment length)	Present study	LA-13	30.2	100.2	29.5	190.5	564.9	2087	672.7	325
	Lee et al. (2016)	LC-13	31.8	105.9	31.0	191.8	561.0	1232	658	260

CHAPTER 6 BOND BEHAVIOUR BETWEEN CNF-LCC AND REINFORCEMENT

3 (short embedding length)	Viwathanate pa et al. (1979)	No. 3	32.5	451.6	63.5	201.3	468.9	2106	737.8	546
	Ueda et al. (1986)	S61	23.8	238.37	59.7	200	438.5	5930	775.7	330
		S107	18.2	754.8	97.4	204	331.6	4626.5	548.2	533
	Engström et al. (1998)	N290b	30.6	200.96	50.24	200	569	921	648	260

(a)

(b)

(c)

(d)

Figure 6.14 Case 1: comparison between analytical and experimental results by Bigaj (1995):

(a) P·16·16·1; (b) P·16·16·2; (c) P·20·16·1; (d) P·20·16·2

(a)

(b)

Figure 6.15 Case 2: comparison between analytical and experimental results by (a) present study; (b) Lee et al. (2016)

(a)

(b)

(c)

(d)

Figure 6.16 Case 3: comparison between analytical and experimental results from pull-out tests:

(a) #3 by Viwathanatepa et al. (1979).; (b) S61 by Ueda et al. (1986); (c) S107 by Ueda et al. (1986); (d) N290b by Engström et al. (1998)

6.4 Summary

Two series of pullout tests were conducted to study the bond behaviour between CNF-LCC/LCC and steel reinforcement at the elastic and post-yielding stage, respectively. The comparable bond strength between CNF-LCC and steel reinforcement compared to NWC was promising and offered similar design treatment for anchorage. The conclusions of CHAPTER 6 are as follows:

1. The shapes of the elastic local bond stress and slip curves of CNF-LCC samples with different bar diameters and LCC samples were similar to those of NWC.
2. When the bar diameter was 16 mm, both CNF-LCC and LCC showed much higher bond strength than traditional foam concrete. The pointwise bond stress of CNF-LCC in the τ -s curves was greater than LCC and the bond strength was increased by 20% due to improved mechanical properties of CNF-LCC. Furthermore, the bond strength of CNF-LCC was almost equivalent to NWC and greater than LWAC for the same compressive strength.
3. The bond strength of reinforcement in CNF-LCC was influenced by the bar diameter. When the diameter was ≤ 16 mm, the bond strength was nearly constant and as high as that in NWC. When the diameter was > 16 mm, the bond strength was significantly reduced by about 30% and it was unlike NWC in which increasing the bar diameter would only slightly reduce the bond strength. Therefore, it is suggested

to use more reinforcing bars of smaller diameter rather than fewer bars of larger diameter in CNF-LCC members.

4. The bond behaviour at the post-yield stage of reinforcement was investigated by increasing the embedment length and the pullout samples showed fracture failure mode although the free end slip was non-zero due to inadequate embedment length. The force-slip relationship obtained in CNF-LCC was similar to that in NWC. In addition, steel strains were measured by strain gauges installed along the long embedment length and corresponding bond stress distribution was computed between adjacent strain gauges at different load stages. The bond strength at the post-yield stage was significantly reduced as a result of reducing cross-section of steel reinforcement and a constant value was assumed to represent it.

5. The concept of control field equations was employed to predict the development length and force-slip relationship of steel bars with different embedment lengths. An analytical model was derived for steel reinforcement with sufficiently long embedment length which consisted of explicit mathematical formulae for perfect anchorage condition. Compared with previous models, the proposed model ensured accuracy by incorporating reasonable constitutive relationships. Besides, the method significantly reduced the computational cost by replacing nested iteration loops with an analytic solution. In the case of insufficient embedment length, a numerical model should be incorporated to solve the non-linear control field equations due to non-zero

free end slip. The predicted results agreed reasonably well with the experimental force-slip relationship from the present study and other publications. Besides, the development length of reinforcement embedded in CNF-LCC was directly obtained by using the proposed models. Generally, the proposed models can be effectively used for analysis and design associated with bond behaviour of reinforced CNF-LCC structures as a result of reduced computational costs and high accuracy.

CHAPTER 6 gives the reliable bond behaviour of reinforcement in CNF-LCC, which established the confidence of using CNF-LCC as a structural material. With all the engineering properties reported so far, it set the stage to investigate the structural performance of reinforced CNF-LCC members which will be reported in the next chapter.

CHAPTER 7 FLEXURAL PERFORMANCE OF REINFORCED CNF-LCC BEAMS

7.1 Introduction

Section 2.2.6 identified that limited research had been conducted on the flexural performance of reinforced foam concrete beams. The studies on the flexural performance of reinforced LWC beams mainly focus on LWAC. The ultimate strength of LWAC beams is practically similar to NWC with the same compressive strength. The method of equivalent rectangular stress block can provide sufficiently accurate predictions (Swamy and Lambert, 1984, Ahmad and Barker, 1991, Evans and Dongre, 1963, Lim, 2007). However, flexural members for structural applications should not only provide adequate strength and sufficient ductility under overload condition for ultimate limit state but must also satisfy serviceability limit state. LWAC beams may have lower stiffness and higher deflection than equivalent NWC beams at serviceability limit state due to lower elastic modulus of LWAC. Evans and Dongre (1963) studied the shear and flexural behaviour of LWAC beams with 1440 ~ 1840 kg/m³ density and 25 ~ 35 MPa strength. The deflections of LWAC beams at service load in their study were 8 ~ 39% higher than NWC beams of the same strength. However, this difference would become smaller when the compressive strength of concrete was increased. Lim (2007) reported that the service load deflections of LWAC with 1860 ~ 1900 kg/m³ density and 35 ~ 65 MPa strength were only 4 ~ 10%

greater than NWC beams. Swamy and Lambert (1984) found that the standard codes significantly underestimated the service load deflections of LWAC. They found that ignoring the effect of tension stiffening can improve the predicted values. Ductility is another important index which represents the ability of structural members to withstand large deformations at approximately the same load after the yielding of steel reinforcement. LWAC beams are expected to exhibit inferior ductility compared to NWC beams because of its more brittle behaviour in terms of material properties. However, research findings on this aspect of LWAC were inconsistent. LWAC beams with compressive strength of 32 ~ 43 MPa from Lim (2007) and of 38 ~ 80 MPa from Ahmad and Barker (1991) showed lower ductility indices than NWC beams. But Murayama and Iwabuchi (1986), as well as Swamy and Lambert (1984) found that the ductility index of LWAC beams was sufficient and comparable with that of equivalent NWC beams when the tension reinforcement ratio was in the low to medium range. Lim (2007) and Wang et al. (1978b) found that the ductility of LWAC beams could be improved with increasing compressive strength, which contradicted the findings from (Ahmad and Barker, 1991, Ahmad and Batts, 1991).

In order to evaluate the structural properties of CNF-LCC based on the observation in previous chapters, a comprehensive test programme of 8 reinforced concrete beams was conducted to study flexural performance of reinforced CNF-LCC beams, in which the variables were content of CNFs, tension reinforcement ratio, compression reinforcement ratio, and steel link ratio. The complete flexural response of reinforced

CNF-LCC beams was investigated including cracking moment and pattern, deflection at service load, ultimate strength, ductility, and full-scale load-deflection relationship. The experimental results were examined by the European requirements for the flexural design and compared with relevant NWC, LWAC and foam concrete collected from the literature. The suitability of using different standard codes to predict flexural response of CNF-LCC beams was evaluated. Finally, an analytical model based on the curvature distribution along the beam length was applied to predict the complete load-deflection relationship of reinforced CNF-LCC beams.

7.2 Experimental programme

7.2.1 Test specimens

The test programme consisted of testing 8 beams with the existence of CNFs, tension reinforcement, compression reinforcement and steel link ratio as the main parameters. All the beams were 150 mm wide, 300 mm deep and 3100 mm long. The design of the beams complied with the requirement in EC 2 (2004). The details of the test beams are shown in Figure 7.1 and Table 7.1. The beam name is one letter followed by three numbers to designate the samples. The letters A, B, C, D, and E represented the concrete type of CNF-LCC, LCC, foam concrete, NWC and LWAC, respectively. The first, second and third numerals denote the respective ratio of tension reinforcement, compression reinforcement and steel link. It should be mentioned that the final five beams listed in Table 7.1 were selected from the tests conducted by Lim (2007) to

compare the reinforced CNF-LCC beam with foam concrete, NWC and LWAC. For the convenient comparison, these five beams from Lim (2007) were renamed by the naming method in this study. The details of beams and the steel reinforcement properties in Lim (2007) were kept the same as Beam A-1.04/0.41/1.85 in this study and the only difference is the concrete material. Beam 23 (C-1.04/0.41/1.85-a) was cast by foam concrete with a target strength of 20 MPa. Beam 25 (C-1.04/0.41/1.85-b) and Beam 26 (C-1.04/0.41/1.85-c) were cast by foam concrete with a target strength of 35 MPa but the water curing age for them was 7 and 28 days, respectively.

A minimum clear distance of 25 mm between individual parallel bars was maintained when different bar sizes were involved. Standard bend method with five times bar diameter length past the end of the bend was utilized to provide anchorage of longitudinal reinforcement. Beam A-1.04/0/2.40 can be considered as singly-reinforced specimen because the compression reinforcement consists of two plain hanger bars with 6 mm diameter. The effect of the transverse reinforcement ratio was studied by varying the spacing of links in the flexural zone. Steel reinforcement with 10 mm diameter was bent into closed links with 135 degrees. A clear concrete cover of 25 mm on all sides of the beams was provided. The links with 130 mm spacing in Beam A-1.04/0.41/1.85 and 100 mm spacing in the other seven beams were designed and provided in the shear zone to ensure flexural failure prior to shear failure. The material properties of longitudinal steel reinforcement used in this study were summarised in Table 7.2.

Figure 7.1 Details of test beams (dimensions are in mm)

Table 7.1 Details of beams from tests and literature

Beam name	Researchers	Type of concrete	Compressive strength (MPa)	Tension bars	Compressive bars	Links in flexural zone	ρ (%)	ρ' (%)	$(\rho - \rho')/\rho_{bal}$ (%)	ρ_{link} (%)
A-1.04/0.41/2.40	Present study	CNF-LCC	32.9	2-H16	2-H10	H10 @ 100	1.04	0.41	0.39	2.40
B-1.04/0.41/2.40		LCC	25.2	2-H16	2-H10	H10 @ 100	1.04	0.41	0.51	2.40
A-0.68/0.41/2.40		CNF-LCC	34.8	2-H13	2-H10	H10 @ 100	0.68	0.40	0.17	2.40
A-1.64/0.41/2.40		CNF-LCC	33.6	2-H20	2-H10	H10 @ 100	1.64	0.41	0.76	2.40
A-1.04/0/2.40		CNF-LCC	36.3	2-H16	2-H6	H10 @ 100	1.04	0.00	0.64	2.40
A-1.04/0.69/2.40		CNF-LCC	32.3	2-H16	2-H13	H10 @ 100	1.04	0.69	0.22	2.40
A-1.04/0.41/3.60		CNF-LCC	35.6	2-H16	2-H10	H10 @ 66.7	1.04	0.41	0.39	3.60
A-1.04/0.41/1.85		CNF-LCC	34.5	2-H16	2-H10	H10 @ 130	1.04	0.41	0.39	1.85
#Beam 23 (C-1.04/0.41/1.85-a)	Lim (2007)	Foam concrete	21.7*	2-H16	2-H10	H10 @ 130	1.04	0.41	0.45	1.85
#Beam 25 (C-1.04/0.41/1.85-b)		Foam concrete	34.5*	2-H16	2-H10	H10 @ 130	1.04	0.41	0.30	1.85
#Beam 26 (C-1.04/0.41/1.85-c)		Foam concrete	37.7*	2-H16	2-H10	H10 @ 130	1.04	0.41	0.30	1.85
#Beam 1 (D-1.04/0.41/1.85)		NWC	42.7*	2-H16	2-H10	H10 @ 130	1.04	0.41	0.28	1.85
#Beam 6 (E-1.04/0.41/1.85)		LWAC	38.1*	2-H16	2-H10	H10 @ 130	1.04	0.41	0.28	1.85

Noting: ρ is tension reinforcement ratio; ρ' is compression reinforcement ratio; ρ_{bal} is balance reinforcement ratio; ρ_{link} is steel link ratio; # represents test beams from (Lim, 2007); * represents 28-days compressive strength based on 100 mm cube;

Table 7.2 Material properties of steel reinforcement

Material properties	Nominal bar size (mm)	Elastic modulus (MPa)	Yield strength (MPa)	Yield strain (%)	Hardening Modulus (MPa)	Ultimate strength (MPa)	Ultimate strain (%)
H13	13	190572	564.9	0.297	2087	672.7	5.468
H16	16	196931	570.5	0.290	1271	674.7	8.511
H20	20	187453	618.7	0.330	1102	717.9	9.340

7.2.2 Preparation and test procedure

All the beam samples were cast in plywood moulds. No vibration process was involved during casting due to self-compacting nature of CNF-LCC & LCC. Sufficient numbers of $\Phi 100 \times 200$ mm cylinders were cast from each batch to determine the mechanical properties of the concrete mix used. The beams and the control specimens were demoulded 24 hours after casting and air-cured at approximately 23 °C in a humidity-controlled chamber room until testing at an age of 28 days.

As shown in Figure 7.2, all the beams were simply supported over a span of 2800 mm and loaded with two symmetrical concentrated loads at the central 800 mm length (flexural zone). The beams were instrumented for measuring deflections at several locations. LVDT 1 was placed at the beam mid-span to measure the deflection. LVDTs 2 and 3 were horizontally instrumented at the top and bottom surfaces of the beam respectively to measure the curvature of the beam over the central gauge length of 450 mm. Besides, concrete and steel strains at critical locations were also determined as shown in Figure 7.1 and Figure 7.2. The development of flexural and shear cracks on the beam surface was recorded at different loading stages. At service load, the number and spacing of flexural cracks were determined and the crack width at the central line of the bottom layer of tension reinforcement was measured by a handheld microscope with 0.02 mm accuracy. The load was applied by a 2000 kN

deflection-controlled hydraulic actuator. The beams were subjected to monotonically increasing load at a rate of 0.3 mm/min up to yielding of tension reinforcement and then the rate was increased to 0.6 mm/min until failure. Applied load, LVDT and strain gauge readings were recorded by a data logger and a computer.

Figure 7.2 Test set-up and instrumentation

7.3 Results and discussion

7.3.1 General behaviour of the beams

Figure 7.3 shows a typical load and mid-span deflection curve of an under-reinforced concrete beam subjected to flexural loading. It consists of four distinct segments separated by five important events, namely, first cracking (I), tension reinforcement yielding (II), initiation of concrete crushing (III), completion of cover spalling (III') and failure of compressive zone (IV). Events I and II result in a reduction of beam

stiffness which is reflected by the reduced curve slope, while events III and III' decrease the load capacity of the beam. Final event IV represents the final crushing of the confined concrete zone by links and the failure load can be assumed to be equal to 85% of the ultimate load (Rashid and Mansur, 2005). Two adjacent events can be connected by a straight line to approximate the curve.

All the beams in this testing programme showed tension failure mode in which tension reinforcement yielded prior to ultimate load and the concrete in compression zone was crushed as shown in Figure 7.4. The experimental load and deflection curves for all the beams are shown in Figure 7.5(a) to Figure 7.8(a) based on the effect of parameters considered in this study. It was found that all the beams exhibited similar load-deflection curves to the typical one although the occurrence of each event and the extent for each segment was dependent on the parameters studied. The moment and curvature curves are shown in Figure 7.5 (b) to Figure 7.8 (b) and their shapes of them are almost the same as the corresponding load-deflection curves. In addition, a comparison of CNF-LCC beam with foam concrete, NWC and LWAC beams from Lim (2007) is presented in Figure 7.9 and Figure 7.10, respectively.

Figure 7.3 Idealized load and deflection curve

(a)

(b)

(c)

(d)

(e)

(f)

(g)

(h)

Figure 7.4 Tension failure modes of (a) Beam A-1.04/0.41/2.40; (b) Beam B-1.04/0.41/2.40; (c) Beam A-0.68/0.41/2.40; (d) Beam A-1.64/0.41/2.40; (e) Beam A-1.04/0/2.40; (f) Beam A-1.04/0.69/2.40; (g) Beam A-1.04/0.41/3.60; (h) Beam A-1.04/0.41/1.85;

(a)

(b)

Figure 7.5 Effect of CNFs on (a) load and deflection curves; (b) Moment and curvature curves

(a)

(b)

Figure 7.6 Effect of tension reinforcement ratio on (a) load and deflection curves; (b) Moment and curvature curves

(a)

(b)

Figure 7.7 Effect of compression reinforcement ratio on (a) load and deflection curves; (b)

Moment and curvature curves

(a)

(b)

Figure 7.8 Effect of links spacing on (a) load and deflection curves; (b) Moment and curvature curves

Figure 7.9 Comparison of load and deflection curves between CNF-LCC and foam concrete from (Lim, 2007)

Figure 7.10 Comparison of load and deflection curves between CNF-LCC, NWC and LWAC from (Lim, 2007)

7.3.2 Cracking moment

The experimental results of the cracking moment of all the beams in this study are listed in Table 7.3. The presence of CNFs increased the cracking moment by about 17% when comparing Beam A-1.04/0.41/2.40 and B-1.04/0.41/2.40. This was attributed to the increased flexural tensile strength of LCC by adding CNFs. In addition, the tension reinforcement ratio was the primary factor to influence the cracking moment while the effect of compression reinforcement ratio and spacing of links was insignificant.

The measured cracking moment was also calculated by the approaches provided by ACI Code (2011), EC 2 (2004) and CEB-FIB Model Code (2010) as listed in Table 7.3. The approaches from all the codes are based on the uncracked elastic flexural theory and the difference among these codes is the predicted expressions of material properties. The tested flexural strength of CNF-LCC and LCC in Section 3.3.2 was used to calculate the cracking moment but all the codes overestimated predictions. The lower experimental cracking moment resulted from the shrinkage of concrete which was restrained by the embedded steel reinforcement (Gilbert, 1999, Ghali, 1993). This restrained shrinkage would produce tension at the extreme fibre of uncracked beam cross section, which resulted in the reduced tensile strength of concrete.

7.3.3 Crack patterns at service load

Service load is the actual load on the structure and it depends on the load types, load combinations and the requirement involved in standard codes. In this study, the service load was assumed to be equal to the ultimate load divided by a factor of 1.6 (Lim, 2007). At service load, the crack patterns including the number of cracks, average spacing of cracks and maximum crack width within the central 800 mm flexural zone of beams are listed in Table 7.4. Comparing Beam A-1.04/0.41/2.40 and B-1.04/0.41/2.40, it can be observed that adding CNFs can improve the crack response in terms of number of cracks, decreasing average crack spacing and the maximum crack width. The better cracking performance can be attributed to the significant tension stiffening effect resulting from the improved bond strength by adding CNFs which was reported in Section 6.3.1. In the tensile zone of a reinforced concrete member, cracking starts when the concrete tensile stress reaches its tensile strength at some points and these cracks are termed as primary cracks. When it occurs, the full tensile force is carried by the tension reinforcement crossing the primary cracks. Between two primary cracks, however, the tensile force is transferred from the steel reinforcement to surrounding concrete by local bond stress and new cracks will appear when the tensile strength of concrete is reached with increasing load. The same stress transfer process will continue between the newly produced cracks until the distance between them is not sufficient to develop the concrete to its tensile capacity and this distance is the spacing of cracks. Higher bond strength of CNF-LCC

with steel bars means a shorter transfer length needed to achieve concrete tensile strength, which resulted in a shorter crack spacing and an increase in the number of cracks compared to LCC. On the other hand, crack width is mainly controlled by the elongation of steel reinforcement on both sides of the considered crack. Increasing bond strength can narrow the crack width because more tensile stress is transferred to surrounding concrete and tensile strain in steel reinforcement can be reduced consequently. The way that reinforcement details influence the crack patterns of reinforced NWC and LWAC beams has been well researched and it can be equally applied to reinforced CNF-LCC beams. Increasing tension reinforcement ratio can effectively reduce the maximum crack width while the effect of compression and transverse reinforcement ratio was relatively insignificant. Cracking problems will influence the appearance, durability and proper functioning of the structure. A limiting crack width of 0.3 mm for serviceability limit state is required in EC 2 (2004) according to the types of structure and their exposure environment. All the beams in this study satisfied this requirement. The maximum crack width of both CNF-LCC and LCC was also significantly lower than traditional foam concrete beams from Lim (2007) in which the crack width greatly exceeded the permissible value by a large margin.

For analytical evaluation, code equations provided in the ACI Code (2011), EC 2 (2004) and CEB-FIB Model Code (2010) were used to calculate the maximum crack width as shown in Table 7.4. It was found that all the codes overestimated the crack

width in this study and the ACI Code gives the best and conservative predictions with the $\omega_{exp}/\omega_{ACI}$ ratio of 0.92. The relatively higher values given by EC 2 and CEB Code may be attributed to underestimation of tensile strength for CNF-LCC.

7.3.4 Stiffness and deflection at service load

Stiffness of beams can be represented by the gradient of load-deflection curves between first cracking (Event I) and tension reinforcement yielding (Event II). As shown in Figure 7.5, the stiffness of Beam A-1.04/0.41/2.40 was slightly greater than Beam B-1.04/0.41/2.40, which was attributed to the increased compressive strength and elastic modulus of concrete by adding CNFs. Figure 7.6 indicates that an increase in tension reinforcement ratio can improve the stiffness of CNF-LCC beams noticeably. However, the compression and transverse reinforcement ratio had practically no influence on the beam stiffness as shown in Figure 7.7 and Figure 7.8. A comparison in Figure 7.9 indicated that the stiffness of CNF-LCC beam is significantly greater than traditional foam concrete beams with 20 and 35 MPa compressive strength. Although the prolonged water curing process was taken to reduce shrinkage in Lim (2007) and improved the stiffness as reflected by the curve of Beam 26 (C-1.04/0.41/1.85-c), CNF-LCC beam still showed a stiffer performance than it due to its higher elastic modulus resulting from the UHPC base mix. In addition, Figure 7.10 shows that the stiffness of CNF-LCC beam was comparable with NWC and LWAC with similar strength. Besides elastic modulus, the high

stiffness of CNF-LCC beams was ultimately attributed to the good bond performance as mentioned in Section 7.3.3. The local bond stress-slip relationship of CNF-LCC reported in Section 6.3.1 is better than LCC and traditional foam concrete, and comparable with NWC and LWAC. Adequate bond strength can cause the effect of tension stiffening and increase the tensile force transfer to concrete between two adjacent cracks, resulting in a higher effective moment of inertia of cracked beam cross-section.

The deflection at service load is closely related to post-cracking stiffness of beams and thus the above discussion can be equally applied to both of them. The mid-span (maximum) deflection measured at service load in this study is presented in Table 7.5. Except for the highest maximum deflection of 12.8 mm in Beam A-1.64/0.41/2.40, the maximum deflection in other 6 CNF-LCC beams ranged from 10.1 to 10.7 mm. LCC beam (Beam B-1.04/0.41/2.40) showed a slightly higher maximum deflection than CNF-LCC beam (Beam A-1.04/0.41/2.40). Table 7.5 also indicates that the mid-span deflection in CNF-LCC beam (Beam A-1.04/0.41/1.85) is slightly higher than NWC beam, comparable with LWAC beam and lower than traditional foam concrete beams. EC 2 (2004) recommends a limiting deflection of $\text{span}/250$ to prevent the damage of appearance. In this study, this limiting value was equal to $2800/250 = 11.2$ mm. Except for Beam A-1.64/0.41/2.40 with a high tension reinforcement ratio, all CNF-LCC beams, as well as NWC & LWAC beam, satisfied this requirement

while LCC beam and foam concrete beams showed higher deflection than this limiting value.

Several methods are provided in standard codes to predict the maximum deflection of flexural beams. To incorporate the effect of tension stiffening, ACI Code (2011) considers an effective moment of inertia which is in between the moment of inertia of uncracked and fully cracked beam cross-section depending on the extent of cracking in the beam, i.e., the ratio of considered load to first cracking load. The deflection is calculated using the elastic flexural theory based on the effective moment of inertia. The methods given by EC 2 (2004) and CEB-FIB Model Code (2010) are basically the same. They calculate the curvature along the length of the beam and then computes the deflection by numerical integration. The assumed curvature lies in between the uncracked and fully cracked conditions and an interpolation coefficient based on the cracking degree is utilized to consider tension stiffening effect. In addition, CEB-FIB Model Code (2010) also provides two simplified methods to predict deflections at serviceability limit state. The first one is using elastic flexural theory to calculate the deflection of uncracked and fully cracked beam, respectively, and then interpolate by the interpolation coefficient. The second one is similar to the one in ACI Code (2011). The predictions from all these methods were compared with experimental results as shown in Table 7.5. It was found that the predictions from all the codes are very close, but they are not conservative when compared with the actual deflections.

A similar conclusion was reported by Rashid and Mansur (2005) when they compared the experimental deflection of high strength concrete beams with the ACI Code predictions. They considered fully cracked beam cross-section, i.e., neglect the tension stiffening effect, and shrinkage-induced tensile stress into the calculations but the improvement was very limited. The shrinkage-induced tensile stress herein was the major factor resulting in the overestimated predictions of the cracking moment in Section 7.3.2. The shrinkage of concrete would be restrained by the embedded reinforcement and it could lead to tensile forces on the concrete at the level of embedded reinforcement. However, the tensile forces would generate an eccentric force if the embedded reinforcement is unsymmetrically placed at the section of an unrestrained (simply supported) concrete beam. Such resulting eccentric force could generate curvature and deflection of significant magnitude with time, even if the beam is unloaded (Gilbert, 2001). All the tested beams in present study were unsymmetrically reinforced and the tension reinforcement ratio was higher than the compression reinforcement ratio. Therefore, the eccentric force could cause the bending of the beam so that the upper beam surface is in compression and the lower one is in tension (Gilbert, 2001). This stress state induced by shrinkage would be sustained in nature until the age of testing and thus the effect of shrinkage-induced creep should be taken into consideration to see if the predictions could be improved. It was recommended to apply the method of effective elastic modulus ($E_{c,eff}$) to

simplify the analysis when involving creep effect caused by external loading as follows (Bazant, 1972):

$$E_{c,eff} = \frac{E_c(\tau_0)}{1 + \chi(t, \tau_0)\phi(t, \tau_0)} \quad (7.1)$$

where $E_c(\tau_0)$ is the concrete elastic modulus at first loading, $\chi(t, \tau_0)$ is the aging coefficient, $\phi(t, \tau_0)$ is the concrete creep coefficient and τ_0 is the first loading age. $\chi(t, \tau_0)$ is a reduced factor for creep coefficient if the loading is applied gradually instead of instantaneous loading. The reported range of $\chi(t, \tau_0)$ was from 0.6 to 0.9 and a constant value of 0.8 is commonly used for long-time creep analysis (Bazant, 1972). The ultimate creep coefficient $\phi(\infty, \tau_0)$ usually ranges from 1.5 to 4.0 when the time reaches infinity (Gilbert, 1998). However, the shrinkage-induced creep is different from the abovementioned creep caused by external loading because the concrete elastic modulus develops gradually from zero at the fresh state to the mature values at the considered ages. Therefore, the aforementioned values of $\chi(t, \tau_0)$ and $\phi(t, \tau_0)$ could not be appropriately applied to the shrinkage-induced creep. Branson (1963) investigated the shrinkage warping of reinforced concrete beam and obtained accurate predictions when employing an effective elastic modulus $E_c/2$ to account for the shrinkage-induced creep effect in the calculations. In other words, the value of $\chi(t, \tau_0)\phi(t, \tau_0)$ was taken as unity in Equation 7.1. Rashid and Mansur (2005) also used the same effective elastic modulus in the deflection calculations of reinforced high strength concrete beams and the accuracy of predictions was

dramatically improved. Therefore, the shrinkage-induced creep effect was also considered in this study and Table 7.6 gives the new predictions using the effective elastic modulus of $E_c/2$. It can be observed that the accuracy of new predictions has been improved significantly. The ratio of experimental results to predictions from all the codes ranged between 0.97 to 1.03 with small standard deviation.

7.3.5 Ultimate strength

The ultimate strength of all the beams is listed in Table 7.7. Although introducing CNFs can improve the compressive strength of LCC by about 30%, the ultimate moment of Beam A-1.04/0.41/2.40 is only slightly greater than that of Beam B-1.04/0.41/2.40 because the relatively lower tension reinforcement ratio in Beam A-1.04/0.41/2.40 and B-1.04/0.41/2.40 impaired the effect of concrete strength. In low-reinforced concrete members, the depth of the compression zone is small and increasing the concrete strength could not increase the level arm significantly resulting in almost unchanged ultimate strength. Table 7.7 indicates that increasing tension reinforcement ratio can significantly increase the ultimate strength while the effect of compression and transverse reinforcement ratio is negligible.

The predicted values of ultimate strength by the standard codes are presented in Table 7.7. The code equations are broadly similar but the expression for compressive stress-strain relationship and maximum compressive strain of concrete are different. ACI Code (2011) considers a simplified rectangular stress block with different sizes

according to the concrete strength. EC 2 (2004) and CEB-FIB Model Code (2010) provide three different stress-strain relationships for calculations including non-linear, parabola-rectangular, and bi-linear relationships. In this study, the most accurate non-linear relationship was employed to predict ultimate strength by the EC 2 (2004) and CEB-FIB Model Code (2010) methods. There is no difference in the maximum strain between NWC and LWAC in ACI Code and that value is 0.003. However, EC 2 and CEB Code consider a lower maximum strain of LWAC than NWC with similar strength. They provide different reduction factors to reflect it and lower density of LWAC will result in lower maximum strain. An attempt was made to measure the maximum compressive strain at the top surface of the concrete by strain gauges in this study and the results for CNF-LCC ranged from 0.023 to 0.0036. As shown in Table 7.7, all the codes gave fairly accurate predictions with small standard deviations. The difference of maximum concrete strain had a negligible effect on the accuracy of predictions, which was the same as in NWC and LWAC. At the same time, good agreement with the codes indicated that CNF-LCC beams have equivalent ultimate strength to NWC and LWAC with similar strength, which was also confirmed by comparing the predictions with the test results in Lim (2007).

7.3.6 Ductility

Ductility can be defined as the ability of a structural member to withstand large deformation without a significant loss in load from the yielding point (Event II) to

the failure point (Event IV). The usual measurement of ductility in a flexural member is at the sectional level based on curvature or at element level based on the deflection. As mentioned in Section 7.2.2, a curvature meter was installed at the central 450 mm of the pure bending span to measure the sectional curvature until the beam failure. However, the curvature meter could only work up to the ultimate strength and subsequent crushing and spalling of concrete cover resulted in inaccuracy in measurements. Therefore, the ductility discussed in this study was based on deflection ductility index μ_d which was equal to the ratio of deflection at the failure of concrete to that at yielding of tension reinforcement. Herein, the failure point was defined as the point in the descending branch of the load-deflection curve with a load equal to 85% of the ultimate load. A minimum ductility index of 3.0 is recommended to allow moment redistribution in design (Park and Paulay, 1975). The deflection ductility indices of all the beams are listed in Table 7.8.

The μ_d of Beam A-1.04/0.41/2.40 is about 40% higher than that of Beam B-1.04/0.41/2.40 because of the increased ductility of CNF-LCC compared to LCC at material level by adding CNFs. The material ductility is closely related to the energy absorption (toughness) of concrete (Moreno et al., 2014). As mentioned in Section 3.3.2 and 3.3.4, the flexural and compressive energy absorption of CNF-LCC was about 51% and 65% higher than those of LCC, respectively. Furthermore, CNF-LCC beam is more influenced by the tension stiffening effect due to better bond-slip relationship, which may also contribute to the improved beam ductility.

One of the usually used methods to guarantee sufficient ductility of the reinforced concrete beam is to limit the tension reinforcement ratio ρ (in the case of a singly reinforced beam) or $\rho - \rho'$ (in the case of a doubly reinforced beam). ACI Code (2011) limits this ratio to $0.75\rho_{bal}$ for common conditions and to $0.5\rho_{bal}$ for structural members in which moment redistribution is considered, where ρ_{bal} is the balanced reinforcement ratio of a singly reinforced beam. The relationship between the μ_d and $\rho - \rho' / \rho_{bal}$ is presented in Figure 7.11. It was found that the ductility of CNF-LCC beam is decreased with increased tension reinforcement ratio. The increased tension reinforcement ratio would increase the neutral axis depth of the beam section and thus the curvature and deflection were reduced, which resulted in reduced ductility of beams. In order to reach the minimum ductility index of 3.0, the maximum value of $\rho - \rho' / \rho_{bal}$ should be 0.6, which was analytically obtained from Figure 7.11. Therefore, the requirement of limited tension reinforcement ratio for NWC in ACI Code (2011) can be equally applied for CNF-LCC. The ductility of beams can be effectively increased by placing compression reinforcement due to the high strength and elastic modulus of steel which can reduce the depth of neutral axis in a flexural beam. In addition, transverse reinforcement can also improve the ductility resulting from the confined effect on the concrete. Figure 7.12 and Figure 7.13 show that increasing both compression reinforcement ratio ρ' or transverse reinforcement ratio ρ_s can effectively increase the ductility index μ_d of CNF-LCC beams. The ductility of NWC, LWAC and traditional foam concrete beams from Lim

(2007) was also compared with CNF-LCC beam and the results are shown in Figure 7.14. When the details of beams and material properties of steel reinforcement were similar, the ductility of CNF-LCC beam was slightly lower than NWC, comparable with LWAC and higher than traditional foam concrete.

Table 7.3 Experimental and predicted results of cracking moment

Beam No.	Cracking moment $M_{c,exp}$ (kN·m)	Predicted cracking moment (kN·m)			Ratio of experimental result to predicted result		
		$M_{c,ACI}$	$M_{c,EC2}$	$M_{c,CEB}$	$M_{c,exp}/M_{c,ACI}$	$M_{c,exp}/M_{c,EC2}$	$M_{c,exp}/M_{c,CEB}$
A-1.04/0.41/2.40	10.5	13.4	13.3	13.7	0.78	0.79	0.77
B-1.04/0.41/2.40	9.0	10.2	10.0	10.4	0.88	0.90	0.87
A-0.68/0.41/2.40	10.0	12.4	12.4	12.6	0.77	0.77	0.75
A-1.64/0.41/2.40	13.5	14.4	14.4	14.9	0.94	0.94	0.91
A-1.04/0/2.40	11.0	13.1	13.0	13.4	0.84	0.85	0.82
A-1.04/0.69/2.40	10.5	13.7	13.6	14.0	0.77	0.77	0.75
A-1.04/0.41/3.60	11.0	13.4	13.3	13.7	0.82	0.83	0.80
A-1.04/0.41/1.85	11.0	13.4	13.3	13.7	0.82	0.83	0.80
Average (standard deviation)					0.83 (0.059)	0.83 (0.061)	0.81 (0.055)

Table 7.4 Crack patterns and predicted maximum crack width

Beam No.	No. of crack	Average crack spacing (mm)	Maximum crack width ω_{exp} (mm)	Predicted maximum crack width (mm)			Ratio of experimental result to predicted result		
				ω_{ACI}	ω_{EC2}	ω_{CEB}	$\omega_{exp}/\omega_{ACI}$	$\omega_{exp}/\omega_{EC2}$	$\omega_{exp}/\omega_{CEB}$
A-1.04/0.41/2.40	14	62	0.18	0.22	0.24	0.25	0.82	0.75	0.72
B-1.04/0.41/2.40	10	83	0.22	0.21	0.24	0.24	1.05	0.92	0.92
A-0.68/0.41/2.40	12	78	0.22	0.22	0.27	0.30	1.00	0.81	0.73
A-1.64/0.41/2.40	13	74	0.16	0.18	0.25	0.23	0.89	0.64	0.70
A-1.04/0/2.40	13	58	0.20	0.21	0.23	0.24	0.95	0.87	0.83
A-1.04/0.69/2.40	14	57	0.18	0.22	0.24	0.25	0.82	0.75	0.72
A-1.04/0.41/3.60	13	64	0.18	0.22	0.24	0.24	0.82	0.75	0.75
A-1.04/0.41/1.85	14	55	0.20	0.22	0.24	0.24	1.00	0.92	0.92
Average (standard deviation)							0.92 (0.094)	0.80 (0.098)	0.79 (0.091)

Table 7.5 Experimental and predicted results of mid-span deflection at service load

Beam No.	Deflection at service load δ_{exp} (mm)	Predicted deflection at service load (mm)				Ratio of experimental result to predicted result			
		δ_{ACI}	$\delta_{EC2(CEB)}$	δ_{CEB-1}	δ_{CEB-2}	$\delta_{exp}/\delta_{ACI}$	$\delta_{exp}/\delta_{EC2(CEB)}$	$\delta_{exp}/\delta_{CEB-1}$	$\delta_{exp}/\delta_{CEB-2}$
A-1.04/0.41/2.40	10.5	9.2	8.9	9.2	9.2	1.14	1.18	1.14	1.14
B-1.04/0.41/2.40	11.3	9.3	9.0	9.3	9.3	1.22	1.26	1.22	1.22
A-0.68/0.41/2.40	10.1	8.0	7.7	8.1	8.1	1.26	1.31	1.25	1.25
A-1.64/0.41/2.40	12.8	11.8	12.0	12.0	12.0	1.08	1.07	1.07	1.07
A-1.04/0/2.40	10.7	9.1	8.9	9.2	9.2	1.18	1.20	1.16	1.16
A-1.04/0.69/2.40	10.5	9.0	8.7	8.9	8.9	1.17	1.21	1.18	1.18
A-1.04/0.41/3.60	10.7	9.1	8.8	9.1	9.1	1.18	1.22	1.18	1.18
A-1.04/0.41/1.85	10.6	9.1	8.8	9.1	9.1	1.16	1.20	1.16	1.16
Average (standard deviation)						1.17 (0.048)	1.21 (0.065)	1.17 (0.050)	1.17 (0.050)
Beam 23 (C-1.04/0.41/1.85-a)	16.9								
Beam 25 (C-1.04/0.41/1.85-b)	17.1								
Beam 26 (C-1.04/0.41/1.85-c)	11.5								
Beam 1 (D-1.04/0.41/1.85)	9.4								
Beam 6 (E-1.04/0.41/1.85)	10.1								

Table 7.6 Predictions of mid-span deflection at service load by considering creep effect

Beam No.	Deflection at service load δ_{exp} (mm)	Predicted deflection at service load (mm)				Ratio of experimental result to predicted result			
		δ'_{ACI}	$\delta'_{EC2(CEB)}$	δ'_{CEB-1}	δ'_{CEB-2}	$\delta_{exp}/\delta'_{ACI}$	$\delta_{exp}/\delta'_{EC2(CEB)}$	$\delta_{exp}/\delta'_{CEB-1}$	$\delta_{exp}/\delta'_{CEB-2}$
A-1.04/0.41/2.40	10.5	11.1	10.5	10.9	10.9	0.95	1.00	0.96	0.96
B-1.04/0.41/2.40	11.3	11.2	10.6	11.1	11.1	1.01	1.07	1.02	1.02
A-0.68/0.41/2.40	10.1	9.5	8.9	9.3	9.3	1.06	1.13	1.09	1.09
A-1.64/0.41/2.40	12.8	14.9	14.2	14.7	14.7	0.86	0.90	0.87	0.87
A-1.04/0/2.40	10.7	11.4	10.8	11.3	11.3	0.94	0.99	0.95	0.95
A-1.04/0.69/2.40	10.5	10.6	10.0	10.2	10.2	0.99	1.05	1.03	1.03
A-1.04/0.41/3.60	10.7	11.0	10.3	10.7	10.7	0.97	1.04	1.00	1.00
A-1.04/0.41/1.85	10.6	11.0	10.4	10.8	10.8	0.96	1.02	0.98	0.98
Average (standard deviation)						0.97 (0.055)	1.03 (0.063)	0.99 (0.060)	0.99 (0.060)

Table 7.7 Experimental and predicted results of ultimate moment

Beam No.	Maximum concrete compressive strain	Ultimate moment $M_{u,exp}$ (kN·m)	Predicted ultimate moment (kN·m)			Ratio of experimental result to predicted result		
			$M_{u,ACI}$	$M_{u,EC2}$	$M_{u,CEB}$	$M_{u,exp}/M_{u,ACI}$	$M_{u,exp}/M_{u,EC2}$	$M_{u,exp}/M_{u,CEB}$
A-1.04/0.41/2.40	0.0023	52.2	53.0	53.2	52.9	0.98	0.98	0.99
B-1.04/0.41/2.40	0.0029	51.4	51.9	52.2	51.6	0.99	0.98	1.00
A-0.68/0.41/2.40	0.0025	37.5	36.2	36.3	36.1	1.04	1.03	1.04
A-1.64/0.41/2.40	0.0027	79.4	84.3	85.3	83.5	0.94	0.93	0.95
A-1.04/0/2.40	0.0036	53.4	53.4	53.7	53.3	1.00	0.99	1.00
A-1.04/0.69/2.40	0.0024	51.7	52.9	53.2	52.9	0.98	0.97	0.98
A-1.04/0.41/3.60	0.0029	53.5	53.4	53.6	53.2	1.00	1.00	1.01
A-1.04/0.41/1.85	0.0028	54.6	53.2	53.5	53.1	1.03	1.02	1.03
Average (standard deviation)						0.99 (0.029)	0.99 (0.031)	1.00 (0.028)

Table 7.8 Experimental results of deflection ductility index

Beam No.	At yielding point		At failure point		Deflection ductility index (δ_f/δ_y)
	Load (kN)	Deflection δ_y (mm)	Load (kN)	Deflection δ_f (mm)	
A-1.04/0.41/2.40	97.5	16.5	88.7	77.4	4.7
B-1.04/0.41/2.40	93.2	16.6	87.3	54.4	3.3
A-0.68/0.41/2.40	66.6	15.0	63.8	96.7	6.4
A-1.64/0.41/2.40	148.7	19.5	135.0	32.9	1.7
A-1.04/0/2.40	94.9	16.2	90.7	45.1	2.8
A-1.04/0.69/2.40	100.5	17.4	87.8	100	5.7
A-1.04/0.41/3.60	93.9	15.3	90.9	101	6.6
A-1.04/0.41/1.85	101.3	17.2	92.8	65.3	3.8
Beam 23 (C-1.04/0.41/1.85-a)	91.3	24.7	83.3	57.9	2.3
Beam 25 (C-1.04/0.41/1.85-b)	96.0	25.1	86.2	58.6	2.3
Beam 26 (C-1.04/0.41/1.85-c)	90.7	16.4	82.2	36.9	2.3
Beam 1 (D-1.04/0.41/1.85)	100.3	15.4	92.7	63.3	4.1
Beam 6 (E-1.04/0.41/1.85)	88.1	14.6	95.8	52.5	3.6

Figure 7.11 Relationship between tension reinforcement ratio $\rho - \rho'/\rho_{bal}$ and ductility index

Figure 7.12 Relationship between compression reinforcement ratio ρ' and ductility index μ_d

Figure 7.13 Relationship between transverse reinforcement ratio ρ_s and ductility index μ_d

Figure 7.14 Comparison of ductility index between CNF-LCC, NWC, LWAC, and traditional foam concrete beams

7.3.7 Prediction of complete load and deflection curve

For the purpose of design and analysis, it is necessary to predict the full-scale load and deflection curves of CNF-LCC beams with various reinforcement design. The

moment and area theorem can be employed based on the curvature distribution along the length of a beam. Unlike an elastic beam, however, the moment and curvature relationship of a reinforced concrete beam is multi-segment because of the occurrence of the five important events as discussed in Section 7.3.1. The same situation of CNF-LCC and LCC beams can be found from the experimental moment and curvature relationship as shown in Figure 7.5(b) to Figure 7.8(b). However, careful observation of these figures showed that the moment-curvature relationship between each event can be assumed to be linear with adequate accuracy. Therefore, the complete moment-curvature relationship of CNF-LCC and LCC beams can be obtained by using straight lines to connect the moment-curvature points of the five successive events, which can be calculated by force equilibrium and Bernoulli's plane strain theory. The measured material properties of CNF-LCC, LCC and steel reinforcement were input into the calculation except for the elastic modulus E_c of CNF-LCC and LCC which was replaced by $E_c/2$ to consider the effect of creep. The EC 2 equation (Equation 3.2) to express the compressive stress-strain curve was used for unconfined CNF-LCC and LCC. However, in absence of experimental results, the expressions of stress-strain curves and maximum strain for confined CNF-LCC and LCC were taken as those recommended by Mansur et al. (1997) and Scott et al. (1989), respectively. A bi-linear stress-strain relationship was utilized for steel reinforcement. According to the computed moment-curvature relationship, the curvature distribution along the length of the beam can be determined at different

loading stages. Therefore, the mid-span deflection can be obtained by the moment area theorem. The main calculation procedure was attached in APPENDIX B and it was processed by a program in Matlab. The analytical and experimental results are compared in Figure 7.15(a) to (h) and it was found that they have good agreement with each other for all 8 beams.

(a)

(b)

(c)

(d)

(e)

(f)

(g)

(h)

Figure 7.15 Comparison between the experimental and analytical load-deflection relationship

7.4 Summary

This chapter investigates the performance of using CNF-LCC in reinforced concrete flexural beams. The experimental results obtained from this study contained all the major findings on its flexural performance. The conclusions of CHAPTER 7 are as follows:

1. Based on the experimental results of compressive stress-strain curves, incorporation of CNF-LCC can improve the elastic modulus, stiffness, maximum strain and compressive energy absorption (toughness). These improvements contributed to the increased stiffness and ductility of CNF-LCC beams.

2. The cracking moment of reinforced CNF-LCC beam was only slightly greater than the LCC beam. But tension reinforcement ratio can significantly influence the cracking moment of CNF-LCC beams.
3. Adding CNFs can improve the crack patterns at service load with more cracks, closer crack spacing and narrower maximum crack width. Only tension reinforcement ratio can affect the crack patterns of CNF-LCC beams. Reasonable predictions for the maximum crack width of CNF-LCC beams can be obtained by the equation from ACI Code.
4. The stiffness of the ascending branch in load-deflection curves was slightly increased by CNFs and thus reduced the deflection at service load. An accurate and conservative estimation of deflection at service load for CNF-LCC and LCC can be made by considering an effective elastic modulus ($E_c/2$) due to creep effect.
5. The methods provided from different codes were found to give accurate predictions for the ultimate load of both CNF-LCC and LCC beams.
6. The ductility of CNF-LCC beam was higher than LCC beam due to the increased ductility of material by adding CNFs. To achieve a ductility index of 3.0, the upper limit of ratio $\rho - \rho' / \rho_{bal}$ should be 0.6 which is required in the ACI Code. The ductility of CNF-LCC beams can be effectively improved by increasing compression and transverse reinforcement ratio.

7. The flexural performance of reinforced CNF-LCC beams was comparable with NWC & LWAC and exceeded traditional foam concrete. In addition, the limitations required in EC 2 for flexural members can be satisfied by CNF-LCC beams, including the maximum crack width and deflection at service load.

8. An analytical method was proposed based on the moment and area theorem to predict the complete load-deflection curves of CNF-LCC and LCC beams, and good agreement with the experimental results was obtained.

CHAPTER 8 CONCLUSIONS AND FUTURE WORKS

8.1 Conclusions

In this research, the potential of CNF-LCC for the structural application was investigated from the material level to the structural level. The mix design optimisation, short-term and long-term properties, and structural performance of CNF-LCC were determined and the experimental results were analysed exhaustively. The significance of findings in this research was apparent to facilitate the design and usage of CNF-LCC for various applications without complicated experiments. The influence of using UHPC base mix and incorporating CNFs was comprehensively studied from the nanostructure of material to the macrostructure of elements, which was absent in previous research work. Besides, the experimental results enriched the limited database for the related engineering properties of foam concrete. Furthermore, the related recommendations and prediction models for CNF-LCC were proposed to guide the design and analysis of reinforced CNF-LCC members in the future. CNF-LCC provided a new solution for structural lightweight concrete. Main conclusions can be drawn from each chapter in this thesis as follows:

In Chapter 3, the mix design of CNF-LCC was customised from that of ceUHPC by adding synthesis forming agent into the latter matrix to create a lightweight cementitious composite with $1500 \pm 50 \text{ kg/m}^3$ density. Optimum particle grading of solid material was used to achieve a higher flow expansion; this was desirable for

micro-foam to be homogeneously blended into the matrix. The mechanical properties of both CNF-LCC & LCC exceeded those of conventional cellular concrete, owing to the usage of ceUHPC/UHPC mix. Incorporating CNFs could modify the microstructure of the base mix and result in different degrees of improvement on mechanical properties. The compressive strength at 1, 7 and 28 days was increased by 18.5%, 16.5%, and 12.7%, respectively. The flexural strength and toughness had the most significant improvement which was 37.1% and 50.8%, respectively. At the same compressive strength level, CNF-LCC showed 39.1% and 71.4% higher flexural strength than NWC and LWAC, respectively. The 28-day elastic modulus was also increased by 15% reaching the equivalent value of LWAC. The 28-day stress and strain relationship indicated that CNFs could increase the maximum strain and compression absorption capacity by 20.8% and 64.8%, respectively. The promising mechanical properties of CNF-LCC showed great potential for structural applications.

In Chapter 4, adding CNFs did not affect the thermal insulation properties, which could be reflected by the TGA and XRD results. Introducing foam bubbles effectively reduced thermal diffusivity and thermal conductivity but did not influence specific heat. The thermal diffusivity of CNF-LCC was lower than NWC and similar to LWAC. The specific heat of CNF-LCC was lower than NWC and LWAC but comparable to HSC due to the lower water content which dominated the value of specific heat. The thermal conductivity of CNF-LCC showed a similar trend but

always lower values than NWC during heating. The factor k , which described the structural efficiency and thermal insulation, of CNF-LCC (0.032) was higher than those of NWC (0.006 ~ 0.019) and LWAC (0.011 ~ 0.029). The good agreement between the one-dimension heat transfer test and numerical results from ABAQUS software verified the measured thermal insulation properties under high temperature. Introducing foam bubbles could reduce thermal shrinkage due to lower cement paste content and adding CNFs could further reduce thermal shrinkage by reducing capillary porosity. The porous structure of CNF-LCC can reduce the internal pressure caused by the generation of CO₂ gas and eliminate thermal expansion after 670 °C. Generally, CNF-LCC showed reasonably good thermal insulation properties and low thermal expansion for fire resistance.

In Chapter 5, low capillary porosity in the ceUHPC base mix and unconnected foam bubbles caused the water penetration depth and water permeability coefficient of CNF-LCC to be 84.6% and 97.6% respectively lower than those of NWC. Besides, CNFs could significantly reduce water penetration and water permeability coefficient of LCC by 45.8% and 70.6%, respectively. This was because the capillary pores that control the permeability of concrete were reduced due to the filler function of CNFs. Although the autogenous shrinkage of CNF-LCC was higher than NWC & LWAC, the total shrinkage of CNF-LCC was lower than them because of the reduced dry shrinkage which can be attributed to the low capillary of UHPC base mix and

shrinkage-reducing nature of the foam agent. CNFs could effectively reduce the autogenous and drying shrinkage via decreasing the volume of critical pores (10-130 nm diameter) by 31% based on the MIP test results. CNF-LCC showed comparable or even better creep resistance than NWC and LWAC with similar compressive strength. Moreover, CNFs effectively reduced creep because they decreased the pathway for movement of gel water to capillary pores and increased the fraction of high-density C-S-H in place of low-density C-S-H. Finally, a precise hyperbolic expression model was proposed to describe the creep development of CNF-LCC and the ultimate creep coefficient was predicted.

In Chapter 6, the pullout tests with short ($5d_b$) and long ($25d_b$) embedment length of steel reinforcement were conducted to study the bond performance of CNF-LCC. In the pullout tests with short embedment length, the bond behaviour between steel reinforcement and CNF-LCC significantly exceeded that of conventional foam concrete. When the bar diameter was ≤ 16 mm, the local bond stress and slip relationship of CNF-LCC was close to NWC and the bond strength was up to 13.9 MPa. When the diameter was > 16 mm, the bond strength of CNF-LCC was reduced by about 30% but still comparable with LWAC. Therefore, it is suggested to use more bars of smaller diameter rather than fewer bars of larger diameter in reinforced CNF-LCC members. In addition, CNFs could increase the bond strength by 20% due to the improved bearing capacity and splitting crack resistance. When increasing the

embedment length to $25d_b$, the force-slip relationship obtained in CNF-LCC was similar to that in NWC and the post-yield bond strength was significantly reduced to about 3.5 MPa due to necking of steel reinforcement. Finally, an analytical model derived by the control field equation with high accuracy and low computing cost was proposed to predict the bond-slip behaviour of different embedment lengths of steel reinforcement in concrete. The reliable bond behaviour of CNF-LCC established the confidence of conducting structural tests on reinforced CNF-LCC beams.

Chapter 7 investigated the flexural performance of reinforced CNF-LCC beams, in which parameters studied included CNFs content, tension and compression reinforcement ratio and steel link ratio. The experimental results showed that the complete flexural response, including cracking moment, crack patterns and deflection at service load, ultimate strength and ductility, of reinforced CNF-LCC beams was comparable with NWC & LWAC and much better than traditional foam concrete. The maximum crack width and deflection of reinforced CNF-LCC beams at service load satisfied the limitations of 0.3 mm and 11.2 mm, respectively, as required in EC 2. In order to ensure the minimum ductility index of 3.0 at ultimate limit states, the maximum tension reinforcement ratio in CNF-LCC beams should be limited to $0.6\rho_{bal}$. Besides, CNFs showed different degrees of improvement on the flexural response especially the ductility of beams which was increased by about 40%. The way that reinforcement details influence the flexural response of reinforced NWC

and LWAC beams can be equally applied to reinforced CNF-LCC beams. Moreover, the prediction methods for the flexural response from different codes were adjusted to be suitable for reinforced CNF-LCC beams. It was worth noting that the shrinkage induced tension and creep should be considered when calculating the cracking moment and deflection at service load, respectively. Finally, an analytical method was proposed based on the moment-area theorem to predict the full-scale load-deflection curves of CNF-LCC beams.

8.2 Future work

In Chapter 3, the mix design of CNF-LCC can be further optimised by introducing supplementary cementitious material to reduce the amount of cement used. Based on the compressive stress-strain test results, micro-fibers such as polypropylene fibers can be considered to modify the brittle behaviour of CNF-LCC.

In Chapter 5, more reinforcement with different diameters should be involved to study their post-yield bond-slip behaviour in CNF-LCC because only $\Phi 13$ mm steel reinforcement was considered in present study. At the same time, the effect of compressive loading, confining reinforcement, bar spacing, transverse pressure and pullout rate should be studied to have a more comprehensive investigation of the bond-slip behaviour between reinforcement and CNF-LCC.

In Chapter 6, the durability of CNF-LCC was evaluated by water penetration depth and converted coefficient of permeability. However, more durability characteristics of CNF-LCC should be measured including diffusion, absorption, gas permeability, chloride permeability, chemical and physical attack.

The structural performance of CNF-LCC was only investigated in the flexural performance of reinforced CNF-LCC beams in Chapter 7. For an extensive application in structures, the shear and combination of shear and flexural performance of reinforced CNF-LCC beams, as well as reinforced CNF-LCC members including slabs, columns and frames should be also studied. Based on the good fire resistance of CNF-LCC presented in Chapter 4, the behaviour of reinforced CNF-LCC elements under high temperature is also worthy to be investigated.

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APPENDIX A CALCULATION PROCESS OF CONTROL FIELD EQUATION MODEL

A.1 Local bond stress-slip relationship

Eligehausen et al. (1982) conducted pull-out tests of shortly embedded reinforcing bars in beam-column joints under monotonic loadings to study the bond resistance mechanism between well-confined normal concrete and deformed steel bars. At the initially low loading stage, the bond resistance is mainly controlled by the chemical adhesion between the steel bar and surrounding concrete and it will disappear when the bar starts to move relatively to surrounding concrete due to increased load. The bond resistance arises from the frictional force and mechanical interlocking between the ribs and concrete. The mechanical interlocking plays a significant role because it provides dominant bond resistance. The bar force can be increased by greater relative slips due to the local concrete crushing in between the bar ribs. However, the radial component of the mechanical locking force also induces tensile hoop stresses which result in splitting cracks in surrounding concrete. If inadequate confinement of concrete is provided, the splitting cracks will propagate to the surface of concrete and the bond resistance will drop to zero. This failure mode is splitting failure which should be avoided in structural members because of low bond strength and brittle behaviour. If the well-confined condition is guaranteed, propagation of splitting cracks can be prohibited and the load can be prevented further. Under such

circumstances, more local crushing of concrete keys in between the ribs takes place, generating more relative slips. The maximum bond resistance is reached when one part of the concrete key is sheared off. With further loading and more slips, the bond resistance will be reduced and only frictional resistance is left since all the concrete keys are sheared off. Finally, the bar will be pulled out. From the test results, the local bond stress-slip relationship could be established to describe bond stress. This model is the most commonly accepted and has been incorporated in the Model Code 2010 (CEB-FIP, 2010) as shown in Figure A.1 and the corresponding mathematical expressions are listed as Equations A.1 to A.4.

Figure A.1 Local bond-slip relationship at elastic state of steel bar (Eligehausen's model)

$$\tau = \tau_m \cdot \left(\frac{s}{s_1}\right)^\alpha \quad \text{when } 0 \leq s \leq s_1 \quad (\text{A.1})$$

$$\tau = \tau_m \quad \text{when } s_1 \leq s \leq s_2 \quad (\text{A.2})$$

$$\tau = \tau_m - \frac{(\tau_m - \tau_f)(s - s_2)}{s_3 - s_2} \quad \text{when } s_2 \leq s \leq s_3 \quad (\text{A.3})$$

$$\tau = \tau_f \quad \text{when } s \geq s_3 \quad (\text{A.4})$$

where τ is the bond stress of reinforcement, τ_m and τ_f are the maximum and frictional bond stress respectively, s is the slip of reinforcement, α , s_1 , s_2 and s_3 are the parameter and slips of reinforcement to define Eligehausen's model.

However, this model is only valid for steel bars exhibiting pullout failure at the elastic stage because of the short embedment length of rebars used in Eligehausen's tests.

Once the bar yields, the rapidly increasing bar strain and the reduced bar cross-section due to Poisson's Ratio will significantly influence the bond behaviour. Several researchers studied the bond behaviour at the post-yield stage of rebars and proposed different local bond-slip relationships with significant discrepancies. Shima *et al.* (1987) conducted pull-out tests of embedded rebars with sufficient long embedment length and deduced the bond stress-strain-slip relationship. In addition, Huang *et al.* (1996) proposed a bilinear local bond-slip relationship with four considered parameters in the model. These models contain more than two variables which are complicated and greatly increase the computational cost. However, Eligehausen *et al.* (1982) and Viwathanatepa *et al.* (1979) proposed a simple but efficient model based on their test results. They found that bond stress was suddenly reduced upon yielding of reinforcement and concrete keys between rebar lugs were sheared off due to inelastic elongation of the bar. This phenomenon conforms to the final branch of Eligehausen's model where the bond stress is provided by friction force (Alsiwat and Saatcioglu, 1992, Pochanart and Harmon, 1989). Therefore, at the post-yield stage of

bars, the bond stress distribution can be assumed as constant with acceptable error. Eligehausen and Viwathanatepa reported that the bond stress of post-yielding bars in tension can be taken as 70-80% of friction bond stress (τ_f) in Eligehausen's model:

$$\tau = \tau_y = 0.7 \sim 0.8 \tau_f \quad (\text{A.5})$$

where τ_y is the bond stress at the post-yield stage of reinforcement. In general, the best way to study the bond resistance is to obtain local bond-slip relationship at both elastic and post-yield states of steel bars.

A.2 Development length

The definition of development length is the shortest embedment length in which the bar stress can increase from zero to the ultimate strength. The calculation diagram is shown in Figure A.2 where the bond stress distribution is curved and constant along the elastic and post-yield segments of the rebar, respectively. When calculating development length, perfect anchorage condition, namely, there is no bar slip at the free end, will be considered. This is a critical boundary condition to solve the control field equation. Table A.1 lists the applied local bond-slip relationships, control field equations, and boundary conditions at the elastic and post-yield segments of the steel bar.

Figure A.2 Calculation diagram of development length of embedded rebar

Table A.1 Control field equations for the elastic and post-yield segments of embedded rebar

Stress state of steel bar	Control field equation	Local bond-slip relationship ($\tau - s$)	Boundary conditions
Elastic segment l_e	Equation 6.4	Equation A.1 ~ A.4	at $x = 0$ $s = s_f = 0$ $s' = \varepsilon_f = 0$
Post-yield segment l_y	Equation 6.5	Equation A.5	at $x = l_e$ $s = s_y$ $s' = \varepsilon_y$

Noting: The term l_e and l_y are the length of the elastic and post-yield state of reinforcement respectively, ε_f is the tensile strain at the free end of reinforcement, s_y is the bar slip at yield point of reinforcement.

A.2.1 Elastic segment of steel bar (l_e)

From Table A.1, the local bond-slip relationship in the elastic segment is Eligehausen's model which consists of four branches. The ascending branch (Equation A.1) is first incorporated into Equation 6.4:

$$\frac{d^2s}{dx^2} - \frac{P_s \tau_m}{EA_s s_1^\alpha} \cdot s^\alpha = 0 \quad \text{when } 0 \leq s \leq s_1 \quad (\text{A.6})$$

Although Equation A.6 is a non-linear second order differential equation, the zero boundary conditions in Table A.1 result in the closed-form solution (Cosenza et al., 2002):

$$s(x) = \left[\frac{P_s \tau_m}{2E A_s s_1^\alpha} \cdot \frac{(1-\alpha)^2}{(1+\alpha)} \right]^{\frac{1}{1-\alpha}} \cdot x^{\frac{2}{1-\alpha}} \leq s_1 \quad (\text{A.7})$$

This solution represents the bar slip distribution along the embedded rebar. Furthermore, the distribution of bond stress $\tau(x)$, bar strain $\varepsilon(x)$, and bar stress $\sigma(x)$ can be obtained according to the local bond-slip relationship, compatibility and stress-strain relationship of the steel bar, respectively.

Because the employed local bond-slip law in the abovementioned solution is the ascending branch of Eligehausen's model, the bar slip at $x = l_e$ should not exceed s_1 . Cosenza et al. (2002) proposed two limited values, namely, the limited bar stress σ_1 and limited embedment length l_1 at $x = l_e$ to satisfy this:

$$\sigma_1 = \sqrt{\frac{2EP_s \tau_m s_1}{A_s(1+\alpha)}} \quad (\text{A.8})$$

$$l_1 = \sqrt{\frac{2EA_s s_1(1+\alpha)}{P_s \tau_m (1-\alpha)^2}} \quad (\text{A.9})$$

It is worth noting that the value of σ_1 is always greater than σ_y , if normal concrete and normal steel bars are used. This suggests that only the ascending branch of Eligehausen's model is responsible for the elastic local bond-slip law at perfect anchorage condition. Meanwhile, the mathematic formulae for the distribution of bar

slip, bar stress, and bond stress are also simplified by these two limited values (Equation A.8 and A.9):

$$s(x) = s_1 \cdot \left(\frac{x}{l_1}\right)^{\frac{2}{1-\alpha}} \leq s_1 \quad (\text{A.10})$$

$$\tau(x) = \tau_m \cdot \left(\frac{x}{l_1}\right)^{\frac{2\alpha}{1-\alpha}} \leq \tau_m \quad (\text{A.11})$$

$$\sigma(x) = \sigma_1 \cdot \left(\frac{x}{l_1}\right)^{\frac{1+\alpha}{1-\alpha}} \leq \sigma_1 \quad (\text{A.12})$$

When the loaded end bar stress σ_l ($\sigma_l \leq \sigma_y < \sigma_1$) is known, the required (shortest) embedment length l_r to develop the bar stress from zero to σ_l at perfect anchorage condition can be derived from Equation A.12:

$$l_r = l_1 \cdot \left(\frac{\sigma_l}{\sigma_1}\right)^{\frac{1-\alpha}{1+\alpha}} \quad (\text{A.13})$$

The elastic segment length l_e can be obtained from Equation A.13 by setting $\sigma_l = \sigma_y$:

$$l_e = l_1 \cdot \left(\frac{\sigma_y}{\sigma_1}\right)^{\frac{1-\alpha}{1+\alpha}} \quad (\text{A.14})$$

A.2.2 Post-yield segment of the steel bar (l_y)

The uniform distribution of bond stress (Equation A.5) is employed at the post-yield segment and the control field equation becomes a linear second order differential equation:

$$\frac{d^2 s}{dx^2} - \frac{P_s}{E_h A_s} \cdot \tau_y = 0 \quad (\text{A.15})$$

The boundary conditions in this segment are the bar slip s_y and bar strain ε_y at the yield point as listed in Table A.1. The value of s_y can be derived from Equation A.10:

$$s_y = s_1 \cdot \left(\frac{l_e}{l_1}\right)^{\frac{2}{1-\alpha}} \quad (\text{A.16})$$

By solving Equation A.15, the closed-form solutions of the distribution of bar slip and bar stress along the post-yielding part are obtained:

$$s(x) = \frac{P_s \tau_y}{2 E_h A_s} \cdot (x - l_e)^2 + \varepsilon_y \cdot (x - l_e) + s_y \quad (\text{A.17})$$

$$\sigma(x) = \sigma_y + E_h \cdot [\varepsilon(x) - \varepsilon_y] = \sigma_y + \frac{P_s \tau_y}{A_s} \cdot (x - l_e) \quad (\text{A.18})$$

The required (shortest) embedment length l_r to develop bar stress from zero to σ_l ($\sigma_l > \sigma_y$) can be derived from Equation A.18:

$$l_r = l_e + \frac{(\sigma_l - \sigma_y) \cdot A_s}{P_s \tau_y} \quad (\text{A.19})$$

Finally, the development length l_d to increase bar stress from zero to the ultimate strength (σ_u) at perfect anchorage condition can be obtained from Equation A.19 by setting $\sigma_l = \sigma_u$:

$$l_d = l_e + \frac{(\sigma_u - \sigma_y) \cdot A_s}{P_s \tau_y} \quad (\text{A.20})$$

A.3 Force-slip relationship

The force-slip relationship is significantly influenced by the embedment length of the rebar. Different embedment lengths exhibit different failure modes and bar stress states at failure. Thus, the prediction of force-slip problems is more complicated than that of development length. The force-slip relationship falls into four different cases according to the embedment length by Kang and Tan (2015) as shown in Table A.2.

Table A.2 Failure modes of embedded bars subject to pull-out force

Case	Embedment length	Free end slip	Bar stress at load end	Failure mode
1	Sufficiently long	Zero	Post – yield	Rupture
2	Long	Non-zero	Post – yield	Rupture
3	Short	Non-zero	Post – yield	Pull-out
4	Extremely short	Non-zero	Elastic	Pull-out

The control field equations will be applied to analyze Case (1) and (3) in this article as there are many related published experimental results to verify the models. Case (2) and (4) are not discussed because the detailed procedure is similar to Case (1) and Case (3).

A.3.1 Case (1) --- “Sufficiently long” embedment length

Case (1) is the same as that introduced in Section A.2 due to perfect anchorage conditions at the free end of the rebar and Figure A.2 can still be used for case study herein. The force–slip relationship will be divided into two phases based on the bar stress state at the loaded end.

A.3.1.1 Loaded end bar stress $\sigma_l \leq \sigma_y$

The required embedment length l_r (Equation A.13) is gradually elongated with increasing applied tensile force. The loaded end slip can be obtained from Equation A.10 when $x = l_r$ and the force-slip relationship can be obtained when $\sigma_l \leq \sigma_y$:

$$F = \sigma_1 A_s \cdot \left(\frac{s}{s_1} \right)^{\frac{1+\alpha}{2}} \quad (\text{A.21})$$

A.3.1.2 Loaded end bar stress $\sigma_y < \sigma_l \leq \sigma_u$

When the applied tensile stress exceeds the yield strength, the required embedment length l_r is further increased as expressed by Equation A.19. The loaded end bar slip can be calculated by setting $x = l_r$ in Equation A.17. Combining Equation A.19 and A.8, a quadratic equation is obtained:

$$\frac{A_s}{2E_h \tau_y P_s} \cdot (\sigma - \sigma_y)^2 + \frac{\varepsilon_y A_s}{\tau_y P_s} \cdot (\sigma - \sigma_y) + s_y - s = 0 \quad (\text{A.22})$$

The following force-slip relationship can be obtained when $\sigma_y < \sigma_l \leq \sigma_u$:

$$F = F_y + \frac{-c_2 + \sqrt{c_2^2 - 4c_1(s_y - s)}}{2c_1} \cdot A_s \quad (\text{A.23})$$

where

$$c_1 = \frac{A_s}{2E_h \tau_y P_s} \text{ and } c_2 = \frac{\varepsilon_y A_s}{\tau_y P_s}$$

Due to the available closed-form solution of control field equations, the force-slip relationship of Case (1) consists of two mathematic formulae (Equation A.21 and A.23) and intensive computation or nested iterations can be avoided.

A.3.2 Case (3) --- “Short” embedment length

The case of “short” embedment length becomes more complicated because perfect anchorage only exists initially with low pull-out force and the free end bar slip will be increased gradually. Meanwhile, both the ascending branch and the plateau branch of Eligehausen’s model are responsible for elastic local bond-slip law. Generally, the force-slip relationship in case (3) can be divided into 5 phases, as summarised in Table A.3 and the calculation diagram for each phase is shown in Figure A.3:

Table A.3 Five phases of force-slip relationship in case (3)

Phases	Free end bar slip s_f	Elastic local bond-slip law($\tau - s$)	Load end bar stress (σ_l)
(a)	Zero	$\tau = \tau_m \cdot (s/s_1)^\alpha$	$\sigma_l < \sigma_y$
(b)	Non-zero	$\tau = \tau_m \cdot (s/s_1)^\alpha$	$\sigma_l \leq \sigma_y$
(c)	Non-zero	$\tau = \tau_m \cdot (s/s_1)^\alpha$	$\sigma_y < \sigma_l < \sigma_u$
(d)	Non-zero	$\tau = \tau_m \cdot (s/s_1)^\alpha$ and $\tau = \tau_m$	$\sigma_y < \sigma_l < \sigma_u$
(e)	Non-zero	$\tau = \tau_m$	$\sigma_y < \sigma_l < \sigma_u$

Figure A.3 Calculation diagram of phase (a), (b), (c), (d) and (e) in Case (3)

A.3.2.1 Phase (a)

Phase (a) is the same as the one discussed in Section A.3.1 due to the same bar stress state and boundary conditions. Therefore, the force-slip relationship in this phase can be obtained from Equation A.21. At the end of this phase, the bond stress propagates to the free end, namely, the bar slip and bond stress are only zero at the free end as shown in Figure A.3 (a).

A.3.2.2 Phase (b)

In this phase, local bond stress–slip relationship is still the ascending branch but the free end bar slip and bond stress are increased gradually. The calculation diagram is shown in Figure A.3 (b). The control field equation remains the same as Equation A.6 but the boundary condition of bar slip at the free end is non-zero which makes it difficult to obtain the closed-form solution. Only numerical solution of Equation A.6 can be obtained by the computer program in Matlab when the free end bar slip is assumed. The assumed free end bar slip is increased gradually until the loaded end bar stress is equal to the yield stress and the force-slip relationship is derived in this procedure. It is necessary to point out that the loaded end bar slip should not exceed s_1 to ensure that the employed elastic local bond-slip relationship is at the ascending branch.

A.3.2.3 Phase (c)

The free end slip in phase (c) will be further increased and the loaded end bar stress goes beyond the yield stress. Therefore, the embedded rebar is divided into the elastic and post-yielding segments as shown in Figure A.3 (c). The same computational procedure as phase (b) is applied to the elastic segment l_e . The position of yield point is located (length of l_e) and the bar slip s_y at the yield point is determined with each assumed free end slip. It should be noted that s_y cannot be higher than s_1 . The

condition in the post-yield segment l_y is the same as Section A.2.2 and the loaded end bar force and slip can be calculated directly:

$$F = F_y + P_s \tau_y \cdot (l - l_e) = F_y + P_s \tau_y l_y \quad (\text{A.24})$$

$$s = \frac{P_s \tau_y}{2E_h A_s} \cdot (l - l_e)^2 + \varepsilon_y \cdot (l - l_e) + s_y = \frac{P_s \tau_y l_y^2}{2E_h A_s} + \varepsilon_y l_y + s_y \quad (\text{A.25})$$

where F_y is the yield force of reinforcement. Phase (c) will be terminated when $s_y > s_1$. Then the plateau branch of Eligehausen's model can be incorporated and the phase is transferred to (d).

A.3.2.4 Phase (d)

Once moving to phase (d), the ascending and plateau branches of Eligehausen's model exist on the elastic segment simultaneously. Therefore, the embedded bar is divided into three segments as shown in Figure A.3 (d). The computational process in phase (b) is employed once again in the first segment l_{e1} . The point where bar slip is equal to s_1 is located (the length of l_{e1}) and the bar strain ε_{e1} as well as bar stress σ_{e1} is calculated at that point with each assumed s_f . In the second segment l_{e2} , the bar stress will be further developed up to the yield stress but the plateau branch is applied herein. The control equation and boundary conditions in the second segment are given by:

$$\frac{d^2 s}{dx^2} - \frac{P_s}{EA_s} \cdot \tau_m = 0; \text{ at } x = l_{e1}, s = s_{e1} = s_1 \text{ and } s' = \varepsilon_{e1} \quad (\text{A.26})$$

The second segment will be terminated at $x = l_e$ where the bar stress is equal to σ_y .

Solving Equation A.26, the value of l_{e2} and s_y are obtained:

$$l_{e2} = \frac{(\sigma_y - \sigma_{e1})A_s}{P_s \tau_m} \quad (A.27)$$

$$s_y = s_1 + \frac{(\varepsilon_y + \varepsilon_{e1})l_{e2}}{2} \quad (A.28)$$

Finally, the loaded end bar force and slip can be calculated in the post-yield segment by Equation A.24 and A.25. Phase (d) is finished when the assumed free end bar slip is equal to s_1 , and after that, only the plateau branch of Eligehausen's model is responsible for the elastic segment of rebar where it is the final phase (e).

A.3.2.5 Phase (e)

In phase (e), the assumed free end bar slip is greater than s_1 and the bond stress distribution is uniform along either elastic or post-yield segment of rebar as shown in Figure A.3 (e). It was found that the stepped bond stress distribution is the same as the assumption in the macro model. The abovementioned analytical methods can be employed in the elastic and post-yield segment. The computational results from Equation A.24 and A.25 show that the loaded end force is constant and bar slip is only governed by the free end slip. Moreover, the maximum applied tensile force capacity is obtained in this phase and it remains until the bar slip at the yielding point exceeds s_2 . As the descending branch of Eligehausen's model is involved if $s_y \geq s_2$,

the force–slip relationship will be moved to unloading segment due to insufficient bond resistance to further develop the bar stress. Therefore, phase (e) is a plateau in the overall force–slip relationship.

APPENDIX B CALCULATION PROCESS OF LOAD AND DEFLECTION RELATIONSHIP

B.1 The stress-strain relationship of confined concrete

The experimental stress-strain curves in Section 3.3.4 showed that unconfined CNF-LCC and LCC only had ascending branch due to their brittle behaviour. However, confined CNF-LCC and LCC can be considered to have the descending branch in the stress-strain relationship based on the beam test results. The compressive stress-strain relationship of confined concrete proposed by Mansur et al. (1997) was recommended in this study as follows:

$$\sigma_c = f_{cm} \left[\frac{\beta \left(\frac{\varepsilon_c}{\varepsilon_{cm}} \right)}{\beta - 1 + \left(\frac{\varepsilon_c}{\varepsilon_{cm}} \right)^\beta} \right] \text{ when } \varepsilon_c \leq \varepsilon_{cm} \quad (\text{B.1})$$

$$\sigma_c = f_{cm} \left[\frac{k_1 \beta \left(\frac{\varepsilon_c}{\varepsilon_{cm}} \right)}{k_1 \beta - 1 + \left(\frac{\varepsilon_c}{\varepsilon_{cm}} \right)^{k_2 \beta}} \right] \text{ when } \varepsilon_{cm} < \varepsilon_c \leq \varepsilon_{cu} \quad (\text{B.2})$$

Where

$$k_1 = 2.77 \left(\frac{\rho_s f_{ys}}{f_{cm}} \right) \quad (\text{B.3})$$

$$k_2 = 2.19 \left(\frac{\rho_s f_{ys}}{f_{cm}} \right) + 0.17 \quad (\text{B.4})$$

$$\beta = 1 / \left(1 - \frac{f_{cm}}{\varepsilon_{c1} E_c} \right) \quad (\text{B.5})$$

$$\varepsilon_{cu} = 0.004 + 0.9\rho_s \left(\frac{f_{ys}}{300} \right) \quad (\text{B.6})$$

σ_c is the concrete stress; ε_c is the concrete strain, f_{cm} is the compressive strength; ε_{cm} is the concrete strain at compressive strength; ε_{cu} is the maximum strain of confined concrete recommended by Scott et al. (Scott et al., 1989); E_c is the concrete elastic modulus; f_{ys} is the yielding strength of transverse reinforcement; ρ_s is the transverse reinforcement ratio.

B.2 Idealized moment and curvature relationship

As mentioned in Section 7.3.7, the idealized moment and curvature relationship of the beam should be obtained by using straight lines to connect the moment-curvature points of the five successive events as shown in Figure B.1. The moment of the five events was marked by M_{cr} , M_y , M_u , M_{sp} , M_f and the corresponding curvature was φ_{cr} , φ_y , φ_u , φ_{sp} , φ_f . The predictions of the points of these five events are calculated as follows:

Figure B.1 Idealized moment and curvature relationship

B.2.1 Event I: First cracking

The moment and curvature at first cracking point can be calculated according to the uncracked elastic flexural theory:

$$M_{cr} = f_r I_g / y_b \quad (\text{B.7})$$

$$\varphi_{cr} = f_r / (y_b E_{c,eff}) \quad (\text{B.8})$$

Where

$$y_b = h - \frac{0.5bh^2 + (n-1) \cdot A_{st} \cdot d + (n'-1) \cdot A_{sc} \cdot d'}{bh + (n-1) \cdot A_{st} + (n'-1) \cdot A_{sc}} \quad (\text{B.9})$$

$$I_g = \frac{1}{12}bh^3 + bh\left(\frac{h}{2} - y_b\right)^2 + A_{st}(n-1)(y_b + d - h)^2 + A_{sc}(n'-1)(h - y_b - d')^2 \quad (\text{B.10})$$

$$n = E_{st}/E_{c,eff} \quad (B.11)$$

$$n' = E_{sc}/E_{c,eff} \quad (B.12)$$

f_r is the flexural tensile strength of concrete; I_g is the moment of inertia for the gross section; y_b is the distance from the neutral axis to the extreme tension fiber of the section; b , h and d are the width, depth and effective depth of the beam cross-section, respectively; d' is the distance between the centroid of the compression reinforcement and extreme compression fiber of the section; $E_{c,eff}$ is the effective elastic modulus of concrete which is equal to $E_c/2$. E_{st} and E_{sc} are the elastic modulus of tension and compression reinforcement, respectively; A_{st} and A_{sc} are the area of tension and compression reinforcement, respectively;

B.2.2 Event II: Tensile reinforcement yielding

When the tension reinforcement is at first yielding strength, the stress in the extreme fiber of the concrete cross-section should be less than the compressive strength f_{cm} because of the under-reinforced design. Therefore, the stress-strain relationship can be approximately assumed as linear and the expressions of M_y and φ_y are:

$$M_y = f_{yt}A_{st} \left(d - \frac{1}{3}kd \right) + E_{sc}\varepsilon_{sc}A_{sc} \left(\frac{1}{3}kd - d' \right) \quad (B.13)$$

$$\varphi_y = \varepsilon_{yt}/(d - kd) \quad (B.14)$$

Where

$$k = \sqrt{(n\rho + (n' - 1)\rho')^2 + 2n\rho + 2(n' - 1)\rho'd'/d} - [n\rho + (n' - 1)\rho'] \quad (\text{B.15})$$

$$\varepsilon_{sc} = \varepsilon_{yt} \cdot (kd - d')/(d - kd) \quad (\text{B.16})$$

f_{yt} and ε_{yt} are the yielding strength and strain of tensile reinforcement, respectively; ε_{sc} is the strain of the compression reinforcement; k is the ratio between the depth of compression zone and effective depth d of the section; ρ and ρ' are the tension and compression reinforcement ratio, respectively;

B.2.3 Event III: Initiation of concrete crush (ultimate strength)

At this stage, the concrete strain at the extreme compression fiber of the section reaches ε_{cm} and the initial crush of concrete occurs. The non-linear stress distribution along the compression depth can be transferred to the rectangular stress block which can be described by α_1 and β_1 . α_1 is the ratio of the uniform stress in the rectangular stress block to the compressive strength f_{cm} while β_1 is the ratio of the rectangular stress block depth to the compression zone depth. α_1 and β_1 can be calculated based on the area (A_{cm}) under the stress-strain curve by Equation B.1 and the first moment (M_{cm}) of this area as follows:

$$A_{cm} = \int_0^{\varepsilon_{cm}} f_{cm} \left[\frac{\beta(\frac{\varepsilon_c}{\varepsilon_{cm}})}{\beta - 1 + (\frac{\varepsilon_c}{\varepsilon_{cm}})^\beta} \right] d\varepsilon_c \quad (\text{B.17})$$

$$M_{cm} = \int_0^{\varepsilon_{cm}} f_{cm} \left[\frac{\beta(\frac{\varepsilon_c}{\varepsilon_{cm}})}{\beta - 1 + (\frac{\varepsilon_c}{\varepsilon_{cm}})^\beta} \right] \varepsilon_c d\varepsilon_c \quad (\text{B.18})$$

$$\alpha_1 = A_{cm} / \left[2 \cdot f_{cm} \cdot \varepsilon_{cm} \cdot \left(1 - \frac{M_{cm}}{A_{cm} \varepsilon_{cm}} \right) \right] \quad (\text{B.19})$$

$$\beta_1 = 2 \left(1 - \frac{M_{cm}}{A_{cm} \varepsilon_{cm}} \right) \quad (\text{B.20})$$

According to the force equilibrium, the following equation can be obtained:

$$A_{st} f_{yt} = A_{sc} E_{sc} \varepsilon_{cm} \cdot (1 - d'/c) + \alpha_1 f_{cm} \beta_1 b c \quad (\text{B.21})$$

The compression depth c of concrete can be obtained by solving Equation B.21 and the values of M_u and φ_u at ultimate strength are:

$$M_u = \alpha_1 f_{cm} \beta_1 b c \cdot (d - 0.5 \beta_1 c) + A_{sc} E_{sc} \varepsilon_{cm} \cdot (1 - d'/c) \cdot (d - d') \quad (\text{B.22})$$

$$\varphi_u = \varepsilon_{cm} / c \quad (\text{B.23})$$

B.2.4 Event III': Completion of cover spalling

Figure B.2 shows the beam cross-section and the strain distribution along the section at this stage. The crushed concrete is represented by the shaded part with a depth of $c_v + d_s$ in Figure B.2. Where c_v and d_s are the concrete cover and diameter of links, respectively. If the compression reinforcement is yielding, $\varepsilon_{sc} \geq \varepsilon_{yc}$, then the compression depth c_1 according to the force equilibrium is:

$$c_1 = (A_{st} f_{yt} - A_{sc} f_{yc}) / (\alpha_1 f_{cm} \beta_1 b) \quad (\text{B.24})$$

f_{yc} and ε_{yc} are the yielding strength and strain of compression steel reinforcement, respectively. The strain of the compression reinforcement can be obtained according to the strain distribution Figure B.2 as follow:

$$\varepsilon_{sc} = \varepsilon_{cm}(1 - 0.5d_c/c_1) \quad (\text{B.25})$$

d_c is the compression reinforcement diameter. If Equation B.25 is not less than ε_{yc} , the moment and curve at Event III' are:

$$M_{sp} = \alpha_1 f_{cm} \beta_1 b c_1 (d - c_v - d_s - 0.5\beta_1 c_1) + f_{yc} A_{sc} (d - d') \quad (\text{B.26})$$

$$\varphi_{sp} = \varepsilon_{cm}/c_1 \quad (\text{B.27})$$

If Equation B.25 is less than ε_{yc} , the compression depth c_1 should be calculated by following the new force equilibrium equation:

$$A_{st} f_{yt} = A_{sc} E_{sc} \varepsilon_{cm} (1 - 0.5d_c/c_1) + \alpha_1 f_{cm} \beta_1 b c_1 \quad (\text{B.28})$$

Therefore, the values of M_{sp} and φ_{sp} are:

$$M_{sp} = \alpha_1 f_{cm} \beta_1 b c_1 (d - c_v - d_s - 0.5\beta_1 c_1) + E_{sc} \varepsilon_{cm} (1 - 0.5d_c/c_1) (d - d') \quad (\text{B.29})$$

$$\varphi_{sp} = \varepsilon_{cm}/c_1 \quad (\text{B.27})$$

Figure B.2 Beam cross-section and strain distribution of the section at the completion of concrete cover

B.2.5 Event IV: Failure of compression zone

The beam cross-section and strain distribution at the failure stage are shown in Figure B.3. Unlike Figure B.2, the shade parts in Figure B.3 stand for the uncrushed concrete. The black and blue shade parts represent the unconfined and confined concrete, respectively. It should be mentioned here that the compression reinforcement at this event is commonly yielding and thus the strain of compression reinforcement in Figure B.3 is equal to ε_{yc} . The stress-strain relationship of confined concrete part is described by Equation B.1 and B.2 and it can be transferred to the rectangular stress block by the similar process as mentioned in Event III.

$$A'_{cm} = \int_0^{\varepsilon_{cm}} f_{cm} \left[\frac{\beta \left(\frac{\varepsilon_c}{\varepsilon_{cm}} \right)}{\beta - 1 + \left(\frac{\varepsilon_c}{\varepsilon_{cm}} \right)^\beta} \right] d\varepsilon_c + \int_{\varepsilon_{cm}}^{\varepsilon_{cu}} f_{cm} \left[\frac{k_1 \beta \left(\frac{\varepsilon_c}{\varepsilon_{cm}} \right)}{k_1 \beta - 1 + \left(\frac{\varepsilon_c}{\varepsilon_{cm}} \right)^{k_2 \beta}} \right] d\varepsilon_c \quad (B.30)$$

$$M'_{cm} = \int_0^{\varepsilon_{cm}} f_{cm} \left[\frac{\beta \left(\frac{\varepsilon_c}{\varepsilon_{cm}} \right)}{\beta - 1 + \left(\frac{\varepsilon_c}{\varepsilon_{cm}} \right)^\beta} \right] \varepsilon_c d\varepsilon_c + \int_{\varepsilon_{cm}}^{\varepsilon_{cu}} f_{cm} \left[\frac{k_1 \beta \left(\frac{\varepsilon_c}{\varepsilon_{cm}} \right)}{k_1 \beta - 1 + \left(\frac{\varepsilon_c}{\varepsilon_{cm}} \right)^{k_2 \beta}} \right] \varepsilon_c d\varepsilon_c \quad (B.31)$$

$$\alpha'_1 = A'_{cm} / \left[2 \cdot f_{cm} \cdot \varepsilon_{cu} \cdot \left(1 - \frac{M'_{cm}}{A'_{cm} \varepsilon_{cu}} \right) \right] \quad (\text{B.32})$$

$$\beta'_1 = 2 \left(1 - \frac{M'_{cm}}{A'_{cm} \varepsilon_{cu}} \right) \quad (\text{B.33})$$

Where A'_{cm} is the area under the stress-strain curve by Equation B.1 and B.2, and M'_{cm} is the first moment of this area; α'_1 is the ratio of the uniform stress in the rectangular stress block of confined concrete to the compressive strength and β'_1 is the ratio of the rectangular stress block depth of confined concrete to the compression zone depth. Based on the force equilibrium and Bernoulli's compatibility, the depth of confined and unconfined concrete zone, c_2 and c_3 are:

$$c_2 = \frac{f_{yt} A_{st} - f_{yc} A_{sc}}{\alpha'_1 f_{cm} \beta'_1 (b - 2c_v - 2d_s) + 2\alpha_1 f_{cm} \beta_1 c_v \varepsilon_{cm} / \varepsilon_{cu}} \quad (\text{B.34})$$

$$c_3 = c_2 \varepsilon_{cm} / \varepsilon_{cu} \quad (\text{B.35})$$

Therefore, M_f and φ_f at failure stage are:

$$M_f = \alpha'_1 f_{cm} \beta'_1 c_2 (b - 2c_v - 2d_s) \left(d - c_v - d_s - \frac{1}{2} \beta'_1 c_2 \right) + 2\alpha_1 f_{cm} \beta_1 c_v c_3 \left(d - c_v - d_s - c_2 + c_3 - \frac{1}{2} \beta_1 c_3 \right) + f_{yc} A_{sc} (d - d') \quad (\text{B.36})$$

$$\varphi_f = \varepsilon_{cu} / c_2 \quad (\text{B.37})$$

Figure B.3 Beam cross-section and strain distribution of the section at the failure of the compression zone

B.3 Load and deflection relationship

The idealized moment-curvature relationship can be obtained based on the calculated M_{cr} , M_y , M_u , M_{sp} , M_f and corresponding φ_{cr} , φ_y , φ_u , φ_{sp} , φ_f . Therefore, the curvature distribution along the length of the beam can be determined at different loading stages. and the mid-span deflection can be obtained by the moment and area theorem. In a simply supported beam under symmetrically loading, the mid-span deflection can be calculated by taking moment, about one support, of the area under the curvature distribution between that support and the mid-span section. The relationship between load F and deflection δ can be obtained from Equation B.38 to B.42:

When $0 \leq \varphi \leq \varphi_{cr}$ and $0 \leq Fa/2 \leq M_{cr}$:

$$\delta = F \cdot \frac{aL^2\varphi_{cr}(3-4\alpha^2)}{48M_{cr}} \quad (B.38)$$

Where a and L are the lengths of shear span and total span of the beam, respectively; α is equal to a/l .

When $\varphi_{cr} < \varphi \leq \varphi_y$ and $M_{cr} < Fa/2 \leq M_y$:

$$\delta = \frac{a^2}{6} \left\{ \varphi_{cr} \left(1 + \frac{2M_{cr}}{Fa} \right) + \left[\frac{Fa(\varphi_y - \varphi_{cr})}{2(M_y - M_{cr})} - \frac{M_{cr}\varphi_y - M_y\varphi_{cr}}{M_y - M_{cr}} \right] \cdot \left(\frac{3}{4\alpha^2} - 1 - \frac{2M_{cr}}{Fa} - \frac{4M_{cr}^2}{F^2a^2} \right) \right\} \quad (B.39)$$

When $\varphi_y < \varphi \leq \varphi_u$ and $M_y < Fa/2 \leq M_u$:

$$\delta = \frac{a^2}{6} \left\{ \frac{4\varphi_{cr}M_y(M_{cr} + M_y)}{F^2a^2} + \varphi_y \left(1 - \frac{2M_{cr}}{Fa} \right) \left(1 + \frac{2M_{cr}}{Fa} + \frac{2M_y}{Fa} \right) + \left[\frac{Fa(\varphi_u - \varphi_y)}{2(M_u - M_y)} - \frac{M_y\varphi_u - M_u\varphi_y}{M_u - M_y} \right] \cdot \left(\frac{3}{4\alpha^2} - 1 - \frac{2M_y}{Fa} - \frac{4M_y^2}{F^2a^2} \right) \right\} \quad (B.40)$$

When $\varphi_u < \varphi \leq \varphi_{sp}$ and $M_{sp} \leq Fa/2 < M_u$:

$$\delta = \frac{a^2}{6} \left\{ \frac{\varphi_{cr}M_y(M_{cr} + M_y)}{M_u^2} + \varphi_y \left(1 - \frac{M_{cr}}{M_u} \right) \left(1 + \frac{M_{cr}}{M_u} + \frac{M_y}{M_u} \right) + \varphi_u \left(2 - \frac{M_y}{M_u} - \frac{M_y^2}{M_u^2} \right) + \left(\frac{3}{4\alpha^2} - 3 \right) \left[\frac{Fa(\varphi_{sp} - \varphi_u)}{2(M_{sp} - M_u)} - \frac{M_u\varphi_{sp} - M_{sp}\varphi_u}{M_{sp} - M_u} \right] \right\} \quad (B.41)$$

When $\varphi_{sp} < \varphi \leq \varphi_f$ and $M_f \leq Fa/2 < M_{sp}$:

$$\delta = \frac{a^2}{6} \left\{ \frac{\varphi_{cr}M_y(M_{cr} + M_y)}{M_u^2} + \varphi_y \left(1 - \frac{M_{cr}}{M_u} \right) \left(1 + \frac{M_{cr}}{M_u} + \frac{M_y}{M_u} \right) + \varphi_u \left(2 - \frac{M_y}{M_u} - \frac{M_y^2}{M_u^2} \right) + \left(\frac{3}{4\alpha^2} - 3 \right) \left[\frac{Fa(\varphi_f - \varphi_{sp})}{2(M_f - M_{sp})} - \frac{M_{sp}\varphi_f - M_f\varphi_{sp}}{M_f - M_{sp}} \right] \right\} \quad (B.42)$$