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STRUCTURAL BEHAVIOUR OF HIGH- STRENGTH RECTANGULAR CONCRETE- FILLED STEEL HOLLOW SECTION COLUMNS

LIU DALIN

**SCHOOL OF CIVIL AND
ENVIRONMENTAL ENGINEERING
NANYANG TECHNOLOGICAL UNIVERSITY**

2006

Structural Behaviour of High-Strength Rectangular Concrete-Filled Steel Hollow Section Columns

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**A thesis submitted to the Nanyang Technological University
in fulfillment of the requirements for the degree of
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LIST OF SYMBOLS

A_c	cross-sectional area of concrete core
A_s	cross-sectional area of steel hollow section
B	breadth of column
D	diameter of circular column
e	load eccentricity
E_c	modulus of elasticity of concrete
E_s	modulus of elasticity of steel
f_0	peak stress of concrete in stress-strain curve
f_c	cylinder strength of concrete (100×200 mm)
f'_c	cylinder strength of concrete (150×300 mm)
f_{cu}	cube strength of concrete (150×150×150 mm)
f_t	tensile strength of concrete
f_y	yield stress of steel
H	height of column ($H \geq B$)
I_c	moment of inertia of concrete core
I_s	moment of inertia of steel hollow section
K	flexural stiffness of column
k	moment amplification factor
K_{ini}	initial flexural stiffness of column
K_{ser}	serviceability-level flexural stiffness
L	length of column
M_0	flexural capacity of column
M_e	experimental ultimate end bending moment
N_c	calculated axial load capacity
N_{buc}	load level at which local buckling occurs
N_{cr}	critical load of slender column
N_e	experimental ultimate axial load
N_{eu}	Euler buckling load
N_u	axial load capacity of stub column
r_s	radius of gyration of steel hollow section
t	wall thickness of steel hollow section

μ	Poisson's ratio
λ	slenderness ratio of column
γ	buckling coefficient
φ	curvature of column or beam
σ_b	local buckling stress
χ	resistance reduction factor due to slenderness effect
ε_0	strain corresponding to peak stress
ε_{bot}	strain in the bottom flange of CFSHS beam
ε_c	strain of concrete
ε_{CR}	strain at geometric centre of cross-section
ε_s	strain of steel
ε_{top}	strain in the top flange of CFSHS beam

SUMMARY

This thesis presents an experimental and analytical investigation of the non-linear behaviour and design of high-strength rectangular concrete-filled steel hollow section (CFSHS) columns under axial loading, pure bending and eccentric loading. The effects of material strength, cross-sectional aspect ratio, slenderness and load eccentricity on the structural behaviour of high-strength rectangular CFSHS columns are extensively examined.

In the experimental programme, 70 stub columns were tested under concentric loading to investigate the axial load behaviour and the ultimate capacity. The concrete of the specimens had compressive strengths ranging from 55 to 106 MPa, whilst the steel sections of the specimens had yield stresses varying from 300 to 550 MPa. The experimental results manifested the strength increase of the concrete due to the confinement provided by the steel hollow section. The strength improvement was found to be dependent on the volumetric steel-to-concrete ratio, the material properties of steel and concrete and the cross-sectional aspect ratio. The tests also showed that the ductility performance of the high-strength concrete was significantly enhanced by the steel hollow section. An analytical model is developed to estimate the axial load behaviour of high-strength rectangular CFSHS stub columns. The calibration of the model shows that it agrees reasonably well with the present experimental results as well as those previously reported by other researchers.

Push-out tests on three specimens were also carried out in the study. The test results suggested that the bond strength of the interface between the steel and the concrete was adversely influenced by the cross-sectional aspect ratio. Rectangular CFSHS columns with higher H/B ratios tended to have lower bond strength.

In order to investigate the flexural behaviour of high-strength rectangular CFSHS columns, 12 specimens were tested in the experimental programme. The specimens were constructed from high-strength materials with steel yield stresses ranging from 409 to 495 MPa and concrete compressive strengths varying from 56 to 91 MPa. Favourable ductility performance with significant yield plateau was

observed for the specimens. The test results of ultimate capacity and flexural stiffness of the specimens are described and considered. An analytical model is developed to evaluate the flexural behaviour of high-strength rectangular CFSHS columns. The calibration suggests that the model can closely predict the flexural strength of the specimens tested in the current study as well as those reported by other researchers.

Additionally, 16 stub columns were tested under eccentric loading to study the effect of load eccentricity on the strength of the columns. The specimens had a uniform steel yield stress of 495 MPa and concrete compressive strength of 60 MPa. The test results showed that the ultimate capacity of high-strength rectangular CFSHS columns was adversely affected by the load eccentricity. On the other hand, 16 slender columns with steel yield stresses of 495 and 550 MPa and concrete compressive strengths of 60 and 71 MPa were tested under eccentric loading to investigate the effects of slenderness and load eccentricity on the strength of the columns. The experiments suggested that both the slenderness and the load eccentricity significantly influenced the ultimate capacity of high-strength rectangular CFSHS columns. An analytical model is constructed to predict the behaviour of high-strength rectangular CFSHS columns subjected to eccentric loading. The verification of the model with the present test data as well as those reported by other researchers suggests that the model can accurately estimate the ultimate capacity of high-strength rectangular CFSHS columns under eccentric loading.

The experimental results obtained in the present study are also employed to calibrate the specifications in the design codes (EC4, ACI and AISC). The comparison between the tests and the codes indicates that modifications should be incorporated into the codes so that a safe and accurate design can be achieved for high-strength rectangular CFSHS columns. Based on the experimental and analytical investigation conducted in this research programme, a set of guidelines are developed for the design of high-strength rectangular CFSHS columns subjected to concentric loading, pure bending and eccentric loading.

CHAPTER 1

INTRODUCTION

1.1 Background

The use of concrete-filled steel hollow section (CFSHS) columns in the construction industry has become increasingly popular in recent years. They are widely used for the construction of building structures, bridges, warehouses and many other structures in earthquake sensitive regions. These columns have demonstrated several advantages over conventional steel hollow section and reinforced concrete columns, such as higher resistance to axial loads, better ductility performance and larger energy absorption capacity.

The enhancement of mechanical properties of CFSHS columns can be attributed to the effect of confinement of the concrete core by the steel hollow section. For an axially loaded CFSHS column, the concrete behaves in a three-dimensional stress state because of the confinement provided by the steel hollow section. On the other hand, the inward local buckling of the steel hollow section is prevented by the concrete infill. The interaction between the steel hollow section and the concrete core results in a considerable increase in strength and resistance to instability of composite columns. Since the steel hollow section can serve as a formwork for the in-filled concrete during construction, the cost saving in terms of manpower and materials can be achieved.

The CFSHS columns used in the construction industry are either circular or rectangular. Circular CFSHS columns are more popular than rectangular ones in building structures, partly because optimum confinement of the concrete by the steel hollow section can be achieved in circular composite columns. Examples include the 72-storey Seg Plaza in China, the 62-storey Gateway Tower in Chicago, the 58-storey Two Union Square in Seattle, the 53-storey First Bank Place in Minneapolis and the 44-storey Pacific First Centre in Seattle. On the other hand,

owing to the ease of fabrication of the beam-to-column connections, rectangular CFSHS columns are becoming widespread in recent years. One example is the Queens Central Tower in Hong Kong. Square CFSHS columns with steel yield stress of 460 MPa were used for the construction of this 292-m-high building.

A significant amount of effort over the past four decades has been aimed at developing a better understanding of the behaviour of CFSHS columns fabricated from normal strength materials. The specifications in the design codes, such as EC4 (1994), ACI (2002) and AISC (1999) have been calibrated against the test data of normal strength CFSHS columns. Thus, the codes are only suitable for the design of normal strength composite columns. For example, EC4 restricts its use to CFSHS columns with steel yield stress (f_y) and concrete cylinder strength (f'_c) lower than 355 and 50 MPa, respectively. It is still unknown if the codes can be reliably used for the design of high-strength CFSHS columns.

With the advent of high-strength steel ($f_y > 400$ MPa) and the production of high-strength concrete ($f'_c > 50$ MPa) using conventional materials, high-strength CFSHS columns are both technically and economically feasible. More architectural and economical benefits can be achieved if high-strength CFSHS columns are used. High-strength concrete has greater damping, stiffness and strength than normal strength concrete. On the other hand, high-strength steel hollow sections provide more confinement to the concrete than mild steel sections if the volumetric steel-to-concrete ratio remains constant. Therefore, high-strength CFSHS columns require a smaller cross-section to withstand the loads, which is very much appreciated by modern designers and building engineers. Despite the advantages, high-strength CFSHS columns are scarcely adopted in the construction industry because little is known on their structural behaviour. A few research projects on high-strength circular and square CFSHS columns have recently been reported by Kilpatrick and Rangan (1999), Varma (2000), Uy (2001a), Varma et al. (2002a, 2002b, 2004), Fujimoto et al. (2004), Inai et al. (2004) and Sakino et al. (2004). Very little has been known about the behaviour of high-strength rectangular CFSHS columns under various types of loading. The current research programme is thus carried out to examine the structural behaviour and to proposed design recommendations for

high-strength rectangular CFSHS columns under axial loading, pure bending and eccentric loading.

1.2 Scope of work

The scope of this thesis is limited to the behaviour of high-strength rectangular CFSHS columns subjected to axial loading, pure bending and eccentric loading. This study includes an extensive experimental programme, in which 117 columns were tested under concentric loading, pure bending and eccentric loading. The effects of material properties, geometric parameters and load eccentricity on the behaviour of high-strength rectangular CFSHS columns were investigated.

In order to provide guidelines for the design of high-strength rectangular CFSHS columns, numerical analyses are conducted based on proposed analytical models. Three types of analytical models are developed for predicting the behaviour of high-strength rectangular CFSHS columns under axial loading, pure bending and eccentric loading. These models are calibrated against the test data obtained in the present experimental programme as well as those reported by other researchers. Based on the experimental and analytical results, recommendations are proposed for the design of high-strength rectangular CFSHS columns under axial loading, pure bending and eccentric loading.

1.3 Objectives and contributions

This thesis deals with the structural behaviour of high-strength rectangular CFSHS columns subjected to axial loading, pure bending and eccentric loading. The objectives of this study are

- (1) To provide a set of test data on the ultimate capacity of high-strength rectangular CFSHS columns subjected to axial loading, pure bending and eccentric loading for the verification of design codes and analytical models;
- (2) To develop constitutive relationships for high-strength concrete confined by rectangular steel hollow sections;
- (3) To propose analytical models for predicting the behaviour of high-strength rectangular CFSHS columns; and

- (4) To recommend guidelines for the design of high-strength rectangular CFSHS columns subjected to axial loading, pure bending and eccentric loading.

1.4 Organisation of thesis

This thesis consists of ten chapters. Chapter 1 gives a brief introduction to the background, scope and objectives of the current research project. The relevant details of the experimental and analytical investigations conducted by other researchers on CFSHS columns are summarised in Chapter 2. The material models reported in the literature for confined concrete are also discussed.

In Chapter 3, the axial load behaviour of high-strength rectangular CFSHS columns was investigated by testing 70 stub columns under concentric loading. The ultimate capacity, load–shortening response and failure mode of the specimens are discussed. Push-out tests on three CFSHS columns were also carried out to investigate the bond-slip behaviour of composite columns.

Chapter 4 presents an analytical study of high-strength rectangular CFSHS stub columns subjected to concentric loading. An analytical model is developed to predict the non-linear axial load behaviour. The proposed model is calibrated against the test data presented in Chapter 3 as well as those reported by other researchers. The provisions in the design codes are also verified with the experimental results.

Chapter 5 reports an experimental study of the flexural behaviour of high-strength rectangular CFSHS columns. A total of 12 specimens were tested under pure bending. Both ultimate capacity and flexural stiffness of the specimens are presented and discussed in this chapter.

Numerical analyses on the flexural behaviour of high-strength rectangular CFSHS columns are subsequently carried out and presented in Chapter 6. An analytical model is proposed for estimating the behaviour of composite columns subjected to pure bending. The test results presented in Chapter 5 as well as those previously reported by other researchers are implemented to verify the proposed model and the specifications in design codes.

Chapter 7 presents an experimental investigation of the behaviour of high-strength rectangular CFSHS columns under eccentric loading. 16 stub columns and

16 slender columns were tested to investigate the effects of material strength, slenderness and load eccentricity on the structural behaviour of the columns. The ultimate capacity, ductility performance and failure mode of the specimens were investigated.

Chapter 8 provides an analytical study of the behaviour of eccentrically loaded high-strength rectangular CFSHS columns. An analytical model is constructed for the numerical analysis. The model is calibrated against the test data obtained in the research programme as well as those reported in the literature. The specifications in the design codes are also verified with the test results for the design of high-strength rectangular CFSHS beam-columns.

Based on the experimental and analytical investigations conducted in the research programme, recommendations are proposed in Chapter 9 for the design of high-strength rectangular CFSHS columns under axial loading, pure bending and eccentric loading. Finally, Chapter 10 presents the conclusions of the research programme. Recommendations for future work on the behaviour of CFSHS columns are also highlighted.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

CFSHS columns have been increasingly popular in recent decades as structural members to resist high compressive loads in the construction industry. These steel–concrete composite columns have manifested many architectural, structural and economical advantages over conventional steel hollow section and reinforced concrete columns. In this chapter, the previous experimental investigations are first reviewed in Section 2.2. The test parameters and research findings in these experimental programmes are summarised. In Section 2.3, the analytical studies undertaken in the past are reviewed. The analytical methods employed and the assumptions made in the studies are discussed in detail. Section 2.4 describes the material models reported by other researchers for simulating the stress–strain relationships of confined concrete. Section 2.5 summarises the literature review and also highlights the originality of the current study.

2.2 Experimental investigation

Since the 1960s, great efforts have been made to investigate the experimental behaviour of CFSHS columns subjected to concentric loading, pure bending, eccentric loading and seismic loading. To ascertain the composite action in composite columns, the creep and shrinkage, bond strength of the interface between the steel and concrete and local buckling of CFSHS columns have also been extensively investigated. These experimental investigations formed the basis for the development of the specifications of the current design codes, such as EC4 (1994), AISC (1999) and ACI (2002).

2.2.1 CFSHS columns subjected to axial loading

One of the first experimental research projects on CFSHS columns was reported by Furlong (1967). Five square and eight circular CFSHS columns were tested under concentric loading. The test results showed that the stability of the steel hollow sections was improved by the concrete infill. However, little strength enhancement resulting from confinement effect was observed. Two years later, Neogi et al. (1969) reported results of eccentric loading tests on circular CFSHS columns with diameter-to-thickness (D/t) ratios ranging from 40 to 80. Based on the test data, a numerical procedure was developed to estimate the strength of axially loaded circular CFSHS columns.

Knowles and Park (1969, 1970) studied seven square and 21 circular CFSHS columns with D/t ratios of 15, 23 and 59. Both short and slender columns with L/D ratios ranging from 2 to 21 were tested. A tangent modulus approach was claimed to conservatively predict the strength of axially loaded CFSHS columns with $L/D > 11$. However, this approach was found to be over conservative for columns with $L/D \leq 11$, since the strength increase of concrete resulting from a confinement effect was significantly underestimated.

Tomii et al. (1977) conducted tests on 268 CFSHS columns to investigate the effects of shape of cross-section, aspect ratio and slenderness on the behaviour of CFSHS columns. Six different sizes of circular, square and octagonal hollow sections with different wall thicknesses were considered. The D/t ratios ranged from 19 to 75, whilst the L/D ratios were between 2 and 9. Two failure modes were observed during the tests. Overall buckling was identified for slender columns, whilst concrete crushing was observed for stub columns. The post-yield behaviour of the CFSHS columns with circular and octagonal cross-sections was either strain hardening or perfectly plastic. On the other hand, a decrease of the applied load was noted in the post-peak region for CFSHS columns with square cross-sections.

Tomii and Sakino (1979a, 1979b) tested 36 short and slender square CFSHS columns. The columns had an identical cross-section of 100×100 mm. Eight short columns were subjected to concentric loading, whilst 28 slender columns were tested under combined axial loading and bending moment. Test results showed that

the axial load level and the breadth-to-thickness ratio significantly affected the axial load behaviour of the columns.

Shakir-Khalil and Zeghiche (1989) investigated the behaviour of seven full-scale rectangular CFSHS columns. The columns were 2760 mm in length and were fabricated from hot-rolled steel rectangular hollow sections ($120 \times 80 \times 5$ mm). The steel yield stresses ranged from 343 to 386 N/mm², whilst the concrete strengths were between 40 and 45 N/mm². Among the specimens, one was tested under axial loading, two were subjected to bending about the major axis, another two were subjected to bending about the minor axis and the remaining two were subjected to biaxial bending. The test results showed that the design code BS5400 (1979) overestimated the strength of the rectangular CFSHS columns subjected to bending about the minor axis.

Shakir-Khalil and Mouli (1990) carried out tests on nine full-scale and 13 short rectangular CFSHS columns under axial loading to determine the ultimate capacity. Three different sizes of steel hollow sections were used (i.e., $120 \times 80 \times 5$, $150 \times 100 \times 5$ and $250 \times 150 \times 6.3$ mm). It was noted that the length-to-height ratios of the short specimens were as low as 1.67 and 1.33. The failure mode of the long columns was identified as overall buckling.

Rangan and Joyce (1992) reported tests on nine circular CFSHS columns under eccentric loading. The yield stress of steel hollow sections was 215 MPa, whilst the strength of concrete was 67 MPa. The effects of slenderness and load eccentricity on the behaviour of columns were studied. A simplified method was presented to calculate the strength of circular CFSHS columns, but the method underestimated the strength of the columns with a maximum discrepancy of 68%.

Schneider (1998) presented an experimental study of the behaviour of concentrically loaded CFSHS stub columns. A total of 14 specimens were tested to investigate the effects of cross-sectional shape and D/t ratio on the ultimate strength of CFSHS columns. The D/t ratios ranged from 17 to 50 and the L/D ratios were between 4 and 5. In the tests, circular CFSCHS columns exhibited better post-yield behaviour and stiffness than rectangular CFSHS columns.

Uy (1998) investigated local and post-local buckling of square CFSHS columns with high D/t ratios. Ten CFSHS columns with D/t ratios between 40 and 100 were

tested. The yield stress of steel hollow sections was 300 MPa. The failure mode, buckling behaviour, residual stress distribution and material constitutive models were discussed in detail. A finite strip method was developed to incorporate elastic and inelastic local buckling of steel hollow sections.

Wang (1999) conducted eccentric loading tests on eight rectangular CFSHS slender columns. The steel hollow sections of the specimens had a uniform cross-section of 120×80×6.3 mm. A design approach was proposed to predict the ultimate capacity of rectangular CFSHS columns.

On the other hand, Kilpatrick and Rangan (1999) conducted experiments on 41 circular CFSHS columns to investigate the effects of load eccentricity and slenderness on the strength of columns. The columns were made from high-strength materials with $f_y = 410\sim 495$ MPa and $f_c = 58$ and 96 MPa. The columns were bent into single or double curvature modes. Test results demonstrated the adverse effects of slenderness and load eccentricity on the strength of the columns.

The experimental behaviour of 18 circular CFSHS columns with large D/t ratios was investigated by O'Shea and Bridge (2000). Test parameters were load condition, D/t ratios ($D/t=60\sim 220$) and material strengths ($f_c=50\sim 120$ MPa, $f_y=190\sim 360$ MPa). Results showed that load condition significantly influenced the axial load behaviour of the columns. Besides, the bond between the steel and the concrete was critical in determining the formulation of local buckling.

Uy (2000) presented an extensive series of tests on square CFSHS stub columns subjected to axial loading combined with bending moment. A total of 30 specimens with $f_y=300$ MPa, $f_c=32\sim 50$ MPa and $B/t=40\sim 100$ were tested. Experiments showed that local buckling was significant in thin-walled composite columns.

Uy (2001a) subsequently reported an extensive set of experiments on 19 square CFSHS columns. Test parameters were B/t ratio ($B/t=20\sim 40$) and material strengths ($f_y=750$ MPa, $f_c=28\sim 32$ MPa). Among the 19 specimens, 10 were concentrically loaded; seven were subjected to eccentric loading; the remaining three were under pure bending. The residual stresses in the steel hollow sections were also investigated. Test results showed that the EC4 approach overestimated the strength of the columns under pure compression and pure bending.

Furthermore, Uy (2001b) investigated the local and post-local buckling behaviour of four square CFSHS columns with $B/t=120\sim180$ and $f_y=265$ MPa. Results illustrated the increase in the critical load of the steel hollow sections due to the in-filled concrete. A post-local buckling model was developed and was found to be conservative to predict the buckling load of the columns.

Han (2002) studied the effects of cross-sectional aspect ratio and material properties on the axial load behaviour of 24 rectangular CFSHS stub columns. Test variables were material strengths ($f_y=194$ and 228 MPa, $f_{cu}=48$ MPa) and cross-sectional aspect ratio ($H/B=1.0\sim1.75$). Test results demonstrated that the axial behaviour of rectangular CFSHS columns was affected by the material properties, cross-sectional aspect ratio and the confinement factor.

Varma et al. (2002a) studied the effects of B/t ratio, steel yield stress and axial load level on the behaviour of square CFSHS beam-columns. Eight specimens with $B/t=32$ and 48 , $f_y=269\sim660$ MPa and $f'_c=110$ MPa were tested under constant axial load combined with increasing bending moment. The failure of the specimens was characterised as local buckling of the steel section and crushing of the concrete. The ACI provisions were found to be accurate for predicting the ultimate capacities of the columns.

Sakino et al. (2004) carried out axial loading tests on 36 circular and 48 square CFSHS stub columns. Test parameters were material strengths ($f_y=262\sim853$ MPa; $f_c=25\sim91$ MPa) and volumetric steel-to-concrete ratio ($0.09\sim0.26$). Tests results indicated a strength increase for square CFSHS columns. However, the strength increase was attributed to the strain hardening of steel section instead of the strength increase of concrete due to the confinement effect.

Fujimoto et al. (2004) reported eccentric loading tests on 33 circular and 32 square CFSHS stub columns. Test variables were material strengths ($f_y=262\sim853$ MPa; $f_c=25\sim85$ MPa), axial load level ($0.15\sim0.80$) and volumetric steel-to-concrete ratio ($0.03\sim0.29$). Ductility improvement due to the confinement of the concrete by the steel hollow section was observed in the tests. For square CFSHS columns, the enhancement of the ultimate capacity due to the confinement effect was found to be minimal.

2.2.2 CFSHS columns subjected to pure bending

In the research project conducted by Furlong (1967), one square CFSHS specimen was tested under pure bending. The specimen was made from normal strength materials ($f_y=331$ MPa, $f'_c=29$ MPa). Test result showed that the ultimate bending moment of the specimen was 1.49 times the plastic bending moment of the steel section. It was therefore concluded that the concrete significantly contributed to the flexural strength of the composite section.

Prion and Boehme (1994) carried out pure bending tests on 26 circular CFSHS specimens with $f_y=262$ MPa, $f'_c=73\sim92$ MPa, $D=152$ mm and $t=1.7$ mm. The two ends of specimens were left uncapped to allow slip between the steel and the concrete to occur. Ductile post-yield performance was observed for all specimens during the tests. Failure of the specimens occurred close to the load points. It was characterised as shear fracture of the concrete followed by stretching and rupture of the steel section. Local buckling was observed in the compression zone of the steel section. Slippages varying from 1 to 2 mm between the concrete and the steel section were observed. The shear span-to-depth ratio showed little effect on the flexural behaviour of the specimens.

Uy (2000) performed pure bending tests on five square CFSHS specimens with $f_y=300$ MPa, $f'_c=29\sim45$ MPa and $B/t=40\sim100$. Ductile post-peak behaviour with significant yield plateau was observed. Furthermore, in another research project by Uy (2001a), three square CFSHS specimens with $f_y=750$ MPa and $f'_c=28\sim32$ MPa were tested under pure bending. The specifications in EC4 were found to be un-conservative to estimate the ultimate capacities of the three specimens. The reason was considered to be the rigid plastic assumptions made in EC4, which assumes fully crushed concrete and fully yielded steel.

Lu and Kennedy (1994) investigated the flexural behaviour of 12 rectangular CFSHS specimens. The specimens were fabricated from normal strength materials ($f_y=350\sim394$ MPa, $f'_c=41\sim47$ MPa). Test variables were cross-sectional aspect ratio and shear span-to-depth ratio. Results showed that the flexural strength of the steel hollow sections was enhanced by the concrete infill by 10~30%. Also enhanced by the concrete infill was the stiffness of the steel section. Moreover, the test results

also showed that slip between the steel and the concrete did not exhibit significant detrimental effect on the flexural behaviour of the specimens.

Elchalakani et al. (2001) reported a set of pure bending tests on 12 circular CFSHS columns with diameters varying from 34 to 110 mm and tube wall thicknesses between 1 and 3 mm. The test results revealed that the EC4 approach accurately predicted the flexural strengths of the specimens.

Han (2004) performed pure bending tests on 16 rectangular CFSHS specimens. The specimens were constructed from normal strength materials ($f_y=294\sim330$ MPa, $f_{cu}=27\sim40$ MPa). The test variables were material strengths, cross-sectional aspect ratio and volumetric steel-to-concrete ratio. The specifications in EC4 and AISC were found to be conservative to predict the ultimate bending moments of the specimens by 20 and 30%, respectively. It is worth noting that EC4 is based on the plastic stress distribution in the composite section to predict the bending resistance. Comments were not given in the study why EC4 considerably underestimated the failure loads of the specimens. The maximum difference between the tests and the EC4 approach was as high as 43%.

2.2.3 CFSHS columns subjected to seismic loading

CFSHS columns have demonstrated better seismic performance than traditional steel hollow section and reinforced concrete columns in many earthquakes, such the Hyogoken-Nanbu Great Earthquake, which occurred in Japan in 1995 (Susantha et al. 2001). A great number of research projects have been carried out since the 1990s on the seismic behaviour of CFSHS columns in wide ranges of geometric and material parameters.

Usami and Ge (1994) reported results of cyclic loading tests on 11 rectangular CFSHS columns. The specimens were constructed from normal strength materials ($f_y=235$ MPa, $f_c=40$ MPa). The cross-sectional aspect ratios were 1.5 and 1.6. Test results showed that the ductility and energy-absorption capacity of the steel hollow sections were enhanced by the in-filled concrete. Additionally, the length and the B/t ratio had adverse effects on the ductility and energy-absorption performance of the columns.

Ge and Usami (1996) further conducted cyclic loading tests on 12 cantilever rectangular CFSHS columns. The specimens were subjected to constant axial loads

($N/N_u=0.20$) combined with cyclic lateral loads. The yield stress of steel was 235 MPa, whilst the cylinder strengths of concrete were between 32 and 39 MPa. The cross-sectional aspect ratios were 1.5 and 1.6. The effects of the length, B/t ratio, number of loading cycles and diaphragm on the seismic behaviour of CFSHS columns were investigated. An analytical model was proposed to predict the capacity of the CFSHS columns.

Elremaily and Azizinamini (2002) performed seismic loading tests on six circular CFSHS columns. Test parameters included axial load level (0.20~0.40), D/t ratio (34 and 51), and material strengths ($f_y=372$ MPa; $f'_c=40\sim104$ MPa). Results indicated that the specifications in AISC were conservative to predict the ultimate capacities of the specimens. The maximum discrepancy was 160%. In the numerical analysis, the concrete model proposed by Mander et al. (1988) was adopted to simulate the confined concrete. The elastic-perfectly plastic stress-strain relationship with von Mises yield criterion was used for the steel hollow section.

Varma et al. (2002b) carried out experimental investigation on the seismic behaviour of high-strength square CFSHS columns. A total of eight specimens with $f_y=269\sim660$ MPa, $f'_c=110$ MPa and $L/B=5$ were tested under axial load levels of 0.10~0.44. The ACI specifications were concluded to be able to accurately predict the ultimate capacities of the specimens. A fibre-based analytical model was developed for predicting the seismic response of the columns. The stress-strain relationship of the confined concrete was derived from three-dimensional analyses of the columns using the finite element software ABAQUS. The analytical model was commented to be capable of estimating the non-linear behaviour of the columns favorably well.

Han et al. (2003) reported tests on 30 normal strength rectangular CFSHS columns subjected to constant axial load combined with seismic bending moment. Test variables were axial load level (0.05~0.69), volumetric steel-to-concrete ratio (0.12~0.17), material strengths ($f_y=280\sim340$ MPa; $f_{cu}=20$ and 61 MPa) and cross-sectional aspect ratio (1.0~2.0). Test results showed that the AISC provisions significantly underestimated the resistance of the specimens to seismic loading with an average difference of 40%.

Inai et al. (2004) performed cyclic loading tests on 13 circular and 20 square CFSHS columns. Test parameters were material strengths ($f_y=400\sim780$ MPa; $f_c=40$ and 90 MPa), depth-to-thickness ratio (23~53) and axial load level (0.30~0.70). Test results showed that composite columns with higher strength of steel and lower depth-to-thickness ratios exhibited better ductility performance. On the other hand, the strength of concrete demonstrated an adverse effect on the cyclic behaviour of the columns.

Varma et al. (2004) carried out cyclic tests on eight square CFSHS columns with $f_y=269\sim660$ MPa and $f'_c=110$ MPa. The columns were subject to constant axial load ($N/N_u=0.10\sim0.22$) combined with cyclically varying bending moment. Results showed that the crushing of concrete and local buckling of the steel section affected the strength and stiffness of the columns. Moreover, the elastic flexural stiffness decreased rapidly due to tension cracking of the concrete and local buckling of the steel hollow section. Besides, the cyclic curvature ductility decreased significantly with an increase of the axial load level. Results also showed that the ACI specifications accurately predicted the moment capacities of the columns.

2.2.4 CFSHS columns subjected to sustained loading

Creep and shrinkage are two important parameters which significantly affect the behaviour of concrete members. A great number of research projects have been carried out to investigate the effects of these two parameters on the behaviour of reinforced concrete columns. It should be noted that CFSHS columns exhibit different behaviour from reinforced concrete columns when subjected to sustained loading. In reinforced columns, concrete experiences shrinkage as it sets during the early age, followed by a lengthy period of creep under sustained loading. However, the concrete in CFSHS columns is expected to undergo a smaller value of shrinkage and creep because the moisture environment is favourably protected inside the steel hollow section. Therefore, creep and shrinkage are expected to have less significant effects on the structural behaviour of CFSHS column than reinforced concrete columns.

Uy (2001c) presented a series of experiments on six square CFSHS columns to ascertain the creep and shrinkage properties of high-strength concrete in CFSHS

columns. Also tested and discussed was the ultimate capacity of the columns after long-term loading. The yield stress of the steel hollow sections was 300 MPa. The cylinder strength of the concrete was 52 MPa. The tests included both concentric and eccentric loading conditions.

Ichinose et al. (2001) reported an experimental study of the creep and shrinkage behaviour of eight circular CFSHS columns. Creep and shrinkage were measured separately in the tests. Plain concrete members were also constructed and tested for comparison. Results suggested that the shrinkage strain of the concrete in composite columns was only 9% of that of the concrete in plain concrete members.

Based on the test results of six square CFSHS columns under sustained loading, Han et al. (2004) commented that shrinkage and creep mainly developed in the first 100 days. Test results showed that the magnitude of creep and shrinkage was adversely affected by the level of the sustained load for slender columns. For short columns, the effect of sustained load level on the creep and strain was found to be insignificant.

2.2.5 Composite action in CFSHS columns

The failure mechanism of the longitudinal bond between the steel and the concrete has been one of the key issues for CFSHS columns. If slip occurs on the interface, the steel section and the concrete will behave independently. The load acting on the column can not be distributed to the concrete by means of bond.

Roeder et al. (1999) compiled a data base of bond strength tests on CFSHS columns, which included 104 circular and 49 square CFSHS specimens. It was concluded that the B/t ratio of the steel section had an adverse effect on the bond strength. A linear regression analysis was carried out to propose a formula to estimate the bond strength of CFSHS columns.

Shakir-Khalil (1993a, 1993b) performed an extensive experimental study of the bond strength of CFSHS columns. A total of 40 stub columns were concentrically loaded to determine the effects of shape of cross-section, size of steel section and condition of the interface on the bond strength of the columns. For some columns, nails were welded to the interior face of the steel section to form shear connectors. Results demonstrated that the shear connectors significantly improved the resistance of the columns to push-out load. It was also concluded that circular CFSHS

columns were more effective than rectangular columns in terms of bond strength. The reason was considered to be the smooth variation of the interface orientation at any position for the circular columns. Similarly, for those specimens with shear connectors, rectangular columns were also observed to be less effective to resist the push-out load than the circular columns.

Table 2.1 summarises the research development of previous experimental investigations on CFSHS columns.

2.3 Analytical investigation

Apart from the experimental investigations mentioned above, a significant amount of effort has also been made in the past 40 years on analytical studies. Owing to different understanding of the structural behaviour of CFSHS columns, various analytical methods have been developed to estimate the behaviour of CFSHS columns. This section summarises the analytical approaches for predicting the ultimate strength of CFSHS columns subjected to combined axial loading and bending moment.

Furlong (1968) proposed that the lower limit of the axial load capacity of a CFSHS column (N_u) was the force that caused the steel hollow section to yield. Besides, the plastic moment of the steel hollow section should be taken as the lower limit of the bending resistance of the composite column (M_0). Hence, the ultimate capacity of the column subjected to combined axial loading with bending moment is determined as

$$(M / M_0)^2 + (N / N_u)^2 \leq 1 \quad (2.1)$$

On the other hand, Knowles and Park (1969) derived a simple straight-line interaction formula to estimate the axial force–bending moment relationship of composite columns, as shown in Equation (2.2). However, the experimental results showed that Equation (2.2) was conservative for short columns but unsafe for slender columns.

$$M / M_0 + P / P_u = 1 \quad (2.2)$$

The Euler approach was developed by Neogi et al. (1969) to estimate the strength of CFSHS columns, as in Equation (2.3). It should be noted that Equation (2.3) is only suitable for predicting the ultimate strength of CFSHS columns within the elastic range of materials. This is because the flexural rigidity of the concrete core decreases in the in-elastic range due to the development of cracks. Thus, the flexural stiffness of the composite column can not be obtained by simply summing up the elastic flexural stiffness of the steel section and the un-cracked concrete core.

$$N_{cr} = \frac{\pi^2 (E_s I_s + E_c I_c)}{L^2} \quad (2.3)$$

in which,

E_s = modulus of elasticity of steel

I_s = moment of inertia of steel hollow section

E_c = elastic modulus of concrete

I_c = moment of inertia of core concrete

L = length of the column

Based on the force and displacement equilibrium, Shams and Saadeghvaziri (1997) proposed Equation (2.4) to calculate the ultimate capacity of slender CFSHS columns subjected to combined axial loading and bending moment.

$$N_{cr} = \phi N_u \quad (2.4)$$

in which ϕ is a function of material strengths, volumetric steel-to-concrete ratio, slenderness ratio and load eccentricity. Once the function is determined, the critical load can be determined by differentiation of Equation (2.4).

Ge and Usami (1994) implemented the finite element software, MARC to determine the strength of CFSHS columns subjected to concentric loading. Link elements were employed in the finite element models to simulate the interaction between the steel hollow section and the concrete.

Schneider (1998) however used another finite element package, ABAQUS to obtain the load–displacement relationship of circular and rectangular CFSHS columns subjected to concentric loading. The concrete was modelled using 20-noded solid elements. The material model available in ABAQUS, called

CONCRETE model was employed to simulate the confined concrete. The uniaxial stress–strain relationship for concrete material was verified by experimental data. The steel section was modelled by 8-noded shell elements. Von Mises yield criterion without strain hardening and Prandtl-Reuss flow rule were adopted for the steel section. The interaction between the steel and the concrete was simulated using gap elements. The coefficient of friction of the interface between the steel and the concrete was assumed to be 0.25.

Shams and Saadeghvaziri (1999) developed 3-dimensional finite element models to simulate the axial load behaviour of CFSHS columns. The concrete was modelled using solid elements. The fracture-based Pramano-Williams model was employed for the confined concrete. This model was calibrated against concrete ($f'_c = 21\sim 41$ MPa) in triaxial compression. The steel section was modelled using shell elements. Von Mises yield criterion without strain hardening and associated flow rule were used for the steel section. The interaction between the steel and the concrete was simulated by uniaxial gap-contact elements. The longitudinal bond and friction were not considered.

In addition to finite element analyses, fibre-based models were also developed (Ge and Usami 1992; Zhang and Shahrooz 1999; Varma et al. 2002b) to evaluate the behaviour of CFSHS columns. This type of method is based on strain compatibility and material constitutive relationships. It treats composite columns as longitudinal steel and concrete fibres. Only the longitudinal stress–strain relationships of the steel and the concrete are considered. Full strain compatibility between the steel section and the concrete is usually assumed.

Shanmugam et al. (2002) proposed an analytical model to predict the local buckling behaviour of thin-walled square CFSHS columns. The residual stresses and geometric imperfections were considered in the analysis by adopting an effective width approach. The analytical model was calibrated against the experimental data reported in previous studies and was found to be fairly accurate in predicting the behaviour and the ultimate load.

Vrcelj and Uy (2002) presented a numerical model to consider the effect of local buckling on the global buckling behaviour of slender CFSHS columns. The B/t ratio and the steel yield stress were found to have significant effects on the local

buckling behaviour of CFSHS columns. An effective width approach was employed in the cross-sectional analysis to account for the local buckling effect.

Furthermore, Mursi and Uy (2003) reported an experimental and theoretical study of thin-walled CFSHS columns. The coupled local and global buckling of CFSHS columns was extensively investigated. A unified model was developed to consider the coupled effects of confinement and local buckling in the elastic and plastic range. The model showed good agreement with the experimental results.

2.4 Material models for confined concrete

Various constitutive relationships for confined concrete have been proposed in the past owing to the complexity of simulating the non-linear behaviour of concrete in various stress states and the difficulty of considering all the factors. Most of the models were proposed for tied and spirally reinforced concrete columns and were later employed for CFSHS columns.

Sargin (1971) suggested the following non-dimensional form for the stress–strain relationship of confined concrete.

$$y = \frac{Ax + Bx^2}{1 + Cx + Dx^2} \quad (2.5)$$

in which, A , B , C and D are parameters controlling the shape of the stress–strain curve,

$$y = \sigma_c / f_0$$

$$x = \varepsilon_c / \varepsilon_0$$

$$\varepsilon_c = \text{strain of concrete}$$

$$\sigma_c = \text{stress of concrete}$$

$$f_0 = \text{peak stress of concrete}$$

$$\varepsilon_0 = \text{strain corresponding to peak stress}$$

Equation (2.5) was later employed by Attard and Setunge (1996) to simulate both the pre-peak and the post-peak behaviour of confined concrete. This model was commented to be suitable for concrete with $f'_c = 20 \sim 130$ MPa. Equation (2.5)

was also implemented by Ahmad and Shah (1982), Yong et al. (1988), Fujimoto et al. (2004) and Sakino et al. (2004).

Popovics (1973) proposed Equation (2.6) to simulate the stress–strain relationship of concrete laterally confined by ties or spirals. This expression was later modified and used by Mander et al. (1988) and Susantha et al. (2000).

$$y = \frac{Ux}{U - 1 + x^U} \quad (2.6)$$

in which, U is a parameter which controls the shape of the stress–strain curve.

Another equation was proposed by Fafitis and Shah (1985) for simulating the post-peak branch of the stress–strain curve of confined concrete, as shown in Equation (2.7). This model was later used by many researchers for the numerical analysis of confined concrete, such as Legeron and Paultre (2003).

$$y = \exp (k_1(\varepsilon_c - \varepsilon_0)^{k_2}) \quad (2.7)$$

where, k_1 and k_2 are two parameters controlling the shape of the stress–strain curve.

For concrete confined by steel hollow sections, a few models have also been proposed. Han (2000) proposed Equation (2.8) for determining the stress–strain response of concrete confined by square steel hollow sections. According to this analytical model, strain hardening can occur if the confinement factor (ξ) is high enough ($\xi > 3.0$). This model was calibrated against test data of square CFSHS columns with $f_y = 194 \sim 750$ MPa and $f'_c = 15 \sim 50$ MPa.

$$\sigma_c = f_o \cdot [A \cdot (\varepsilon_c / \varepsilon_0) - B \cdot (\varepsilon_c / \varepsilon_0)^2] \quad \text{for } \varepsilon_c \leq \varepsilon_0 \quad (2.8a)$$

$$\sigma_c = \frac{f_o \cdot (\varepsilon_c / \varepsilon_0)}{\beta \cdot (\varepsilon_c / \varepsilon_0 - 1)^\eta + \varepsilon_c / \varepsilon_0} \quad \text{for } \varepsilon_c > \varepsilon_0 \quad (2.8b)$$

in which,

$$f_o = f_{ck} \cdot [1.194 + 0.25 \cdot (13 / f_{ck})^{0.45} \cdot (-0.07845 \cdot \xi^2 + 0.5789 \cdot \xi)] \quad (2.9)$$

$$\varepsilon_o = 1300 + 14.93 f_{ck} + 0.95 \cdot [1400 + 800 \cdot (\frac{f_{ck} - 20}{20})] \cdot \xi^{0.2} \quad (2.10)$$

$$\eta = 1.60 + 1.5 \cdot (\varepsilon_o / \varepsilon_c) \quad (2.11)$$

$$A = 2.0 - 0.1 \cdot \xi^{0.745} \quad (2.12)$$

$$B = 1.0 - 0.1 \cdot \xi^{0.745} \quad (2.13)$$

$$\beta = \frac{0.75 \cdot f_{ck}^{0.1}}{\sqrt{1 + \xi}} \quad \text{for } \xi \leq 3.0 \quad (2.14a)$$

$$\beta = \frac{0.75 \cdot f_{ck}^{0.1}}{\sqrt{1 + \xi(\xi - 2)^2}} \quad \text{for } \xi > 3.0 \quad (2.14b)$$

$$\xi = A_s f_y / A_c f_{ck} , \quad \text{confinement factor} \quad (2.15)$$

$$f_{ck} = 0.67 f_{cu} = 0.85 f_c' \quad \text{in MPa} \quad (2.16)$$

2.5 Concluding remarks

The literature review on previous research projects pertaining to CFSHS columns under various types of loading has been presented. Both experimental and analytical investigations on the behaviour of CFSHS columns are considered. The material models reported by other researchers for simulating confined concrete are also reviewed in this chapter.

From the literature review, it can be concluded that a great many experimental and analytical investigations have been carried out to evaluate the behaviour of CFSHS columns. However, the composite action between the steel and the concrete in CFSHS columns is still not well understood. Furthermore, the research information on the behaviour of high-strength CFSHS columns, especially high-strength rectangular CFSHS columns is insufficient. As a result, the use of high-strength rectangular CFSHS columns in the construction industry has been restricted. Therefore, there is a research need to carry out an experimental and analytical study to achieve a better understanding of the behaviour and a more accurate design of high-strength rectangular CFSHS columns.

Table 2.1 Previous experimental investigations of CFSHS columns

Con- crete	Steel	Cross- section	f'_c (MPa)	f_y (MPa)	Static loading		Cyclic loading	Sus- tained loading	Columns	
					Con- centric	Eccen- tric			Short	Slender
<i>N</i>	<i>N</i>	○	20~50	215~390	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>
		□	20~50	215~345	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>
		□	24~50	228~386	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>
<i>N</i>	<i>H</i>	○	20~50	413~537	<i>Yes</i>	<i>No</i>	<i>No</i>	<i>No</i>	<i>Yes</i>	<i>No</i>
		□	20~50	485~835	<i>Yes</i>	<i>Yes</i>	<i>No</i>	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>
		□	26~50	413~430	<i>Yes</i>	<i>No</i>	<i>No</i>	<i>No</i>	<i>Yes</i>	<i>No</i>
<i>H</i>	<i>N</i>	○	60~112	200~360	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>
		□	80~110	262~269	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>
		□	50~60	228~370	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>No</i>	<i>Yes</i>	<i>Yes</i>
<i>H</i>	<i>H</i>	○	68~90	400~780	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>No</i>	<i>Yes</i>	<i>Yes</i>
		□	70~110	618~835	<i>Yes</i>	<i>Yes</i>	<i>Yes</i>	<i>No</i>	<i>Yes</i>	<i>Yes</i>
		□	50~100	400~800	<i>No</i>	<i>No</i>	<i>No</i>	<i>No</i>	<i>No</i>	<i>No</i>

Notes:

Yes: Research work has been conducted; *No*: Research work has not been conducted*N*: Normal-strength material; *H*: High-strength material

CHAPTER 3

EXPERIMENTAL STUDY OF RECTANGULAR CFSHS COLUMNS UNDER CONCENTRIC LOADING

3.1 Introduction

This chapter presents the results of three experimental programmes which were carried out to investigate the axial load behaviour of high-strength rectangular CFSHS stub columns. A total of 73 stub columns were tested at three universities, i.e. Nanyang Technological University (NTU), Harbin Institute of Technology (HIT) and Fuzhou University (FU). The failure loads of stub columns are used to establish an upper limit on the resistance of slender CFSHS columns to axial loads. Once defined, the ultimate capacity of CFSHS columns can be determined by incorporating the effects of slenderness, load eccentricity, geometric and material imperfections, etc.

The length-to-height (L/H) ratio of all specimens tested in the present study was determined to be 3.0, following the suggestions made by Zhong (1994), Uy (2001a), Han (2002) and Sakino et al. (2004). For columns with $L/H=3$, the effect of slenderness on the behaviour of CFSHS columns was considered to be negligible.

The steel hollow sections of CFSHS columns used in the construction industry are either hot-rolled, cold-formed or welded sections. The hot-rolled hollow sections usually have larger width-to-thickness ratios than cold-formed ones. Hence, the hot-rolled hollow sections may be uneconomical in some cases. The cold-form work may induce residual stresses and strain hardening of the steel material at the corners of the cross-section. The corner effects need to be considered in the determination of ultimate capacity of CFSHS columns using cold-formed hollow sections. The hollow sections used for the stub columns tested in the current study were fabricated by welding the four component plates. When fabricating this type of hollow sections, residual stresses are inevitably induced in the welding process.

However, the effect of residual stresses on the axial load capacity of CFSHS stub columns with compact cross-sections is minimal, because the residual compressive force caused by welding is equal in magnitude to the residual tensile force in the cross-section. For CFSHS columns with high H/t ratios, the residual stresses resulting from welding can significantly affect the local buckling behaviour of the column and therefore influence the axial load capacity.

3.2 Experimental programme at NTU

A total of 17 specimens were constructed and tested at NTU. 14 of the specimens were tested to failure under concentric loading to determine the ultimate capacity, whilst the remaining three were for push-out tests to investigate the bond-slip behaviour. The primary test parameters were material strengths ($f_y=300$ MPa, $f'_c=83$ and 106 MPa), cross-sectional aspect ratio (1.0~1.8) and volumetric steel-to-concrete ratio (0.13~0.24). All the specimens were fabricated from welded steel hollow sections and high-strength concrete infill, as shown in Fig. 3.1.

3.2.1 Material properties

In order to obtain the actual material properties of the steel sections, three coupons were machined according to the requirements in ASTM (1997). The dimensions of the coupons are shown in Fig. 3.2 (a). The coupons were tested in an Instron testing machine in the Structural Mechanics Laboratory of NTU. The loading rates were 0.5 mm per minute before the yield load and 1.5 mm per minute after the yield load. An extensometer with gauge length of 50 mm was attached to the centre of the coupons to record the elongation. It was noted that the readings from the extensometer were of great significance for plotting the post-yield branch of the stress-strain curves. This was because, as shown by the current tests, the strain gauges normally were unable to give the correct readings beyond a tensile strain of 0.01. The test setup and the failure modes of the steel coupons are shown in Fig. 3.3. A typical stress-strain curve of the coupons is illustrated in Fig. 3.4(a). The average yield stress and modulus of elasticity were determined as 300 and 203,000 MPa, respectively. Table 3.1 summarises the results of the tension tests.

The concrete for the composite columns was mixed in two batches with different mix proportion details. From each batch of concrete, three cubes (150×150×150 mm) and three cylinders (150×300 mm) were prepared. In order to simulate the actual curing condition of the concrete inside the steel hollow section, the concrete samples were cured in the moulds covered by wet hessian for seven days and were placed in a fog room thereafter to maintain the temperature and humidity. At the same day as the CFSHS specimens were tested, the concrete cubes were tested in a 3000-kN-capacity machine at a loading rate of 330 kN per minute, as shown in Fig. 3.5. For the concrete cylinders, both ends were ground flat and smooth in a grinding machine and were then tested in a 3000-kN-capacity testing machine at a loading rate of 265 kN per minute, as shown in Fig. 3.6. The details of the mix proportions for the two batches of concrete and the test results of the concrete samples are summarised in Table 3.2.

3.2.2 Fabrication of specimens

The steel hollow section was fabricated by welding its four component flat plates together. A 10-mm-thick cap plate was welded to its base to support the wet concrete during construction. The concrete was then vertically cast into the steel hollow section in three layers. Each layer of concrete was compacted using a poker vibrator. A 10-mm gap was left from the concrete surface to the top of the steel section. The concrete was cured inside the steel hollow section for 21 days. As the concrete shrank over time, a layer of high-strength cement mortar was in-filled to flush the concrete core with the steel section. Finally, a 10-mm-thick cap plate was welded to the top to form a complete specimen. Fig. 3.7 illustrates the fabrication process of the specimens. The geometric and material properties of the specimens are summarised in Table 3.3.

3.2.3 Test procedure and results

The tests were carried out in a 5000-kN-capacity Instron testing machine in the Heavy Structures Laboratory of NTU. The details of the test setup are depicted in Fig. 3.8. In order to measure the axial shortening of the specimen during the test, four displacement transducers were mounted symmetrically around the specimen between the two platens of the testing machine. Eight electric resistance strain

gauges were bonded at mid-height with two on each face of the steel hollow section to record the longitudinal and transverse strains.

Prior to the actual test, a pre-load of 100 kN was applied to the specimen three times to ensure the effectiveness of strain gauges. A load level of 20 kN was subsequently applied so that the platens of the testing machine were firmly attached to both ends of the specimen. Then the axial load was applied in a displacement-control method at a loading rate of 0.3 mm per minute. During the test, the longitudinal and the transverse strains as well as the axial shortening of the specimens were recorded at a load increment of 50 kN. The test for each specimen took approximately 45 minutes to complete.

Outward local buckling was observed after the peak load, but no buckling occurred before the peak load for all specimens. This demonstrated that the composite sections were compact before the failure load. The experimental failure loads (N_e) and the loads (N_{buc}) at which visible buckling occurred are summarised in Table 3.3.

All specimens exhibited favourable ductility performance during the tests. As shown in Fig. 3.9, the descending branch of the load–shortening curves was smooth and stable. Large deformations were recorded for all columns without crushing of concrete. By comparison, abrupt crushing would occur for unconfined high-strength concrete at the peak load. Post-yield behaviour with large deformation would not be expected for the un-confined concrete. Thus, it was concluded that the ductility performance of the high-strength CFSHS columns was significantly enhanced due to the confinement effect.

Fracture of the longitudinal welds occurred for Specimens A3-2, A4-1 and A8-2, as shown in Fig. 3.10(a). Tests were terminated upon the failure of welds. For the other specimens, tests were terminated when the load–shortening curves had reached a plateau.

The failure of the specimens was characterised as outward local buckling of the steel hollow section. However, the local buckling mode was found to be influenced by the cross-sectional aspect ratio. As shown in Fig. 3.10(b), local buckling occurred almost to the same extent and simultaneously on each face for specimens with $H/B = 1.0$. However, for specimens with $H/B = 1.3$ and 1.8 , buckling was more

extensive and occurred earlier on the broader faces. This observation revealed that rectangular steel hollow sections with high aspect ratios would be ineffective to resist the load, since the effective area of the steel section which confined the concrete was reduced.

3.2.4 Push-out tests

Push-out tests were conducted to investigate the bond-slip behaviour of CFSHS columns and to obtain the bond strength. Three specimens (Specimens A9-1, A9-2 and A10) were prepared and subjected to push-out load in the 5000-kN-capacity Instron testing machine. As shown in Fig. 3.11, four displacement transducers were placed symmetrically around the specimen to measure the slippage. A steel block was placed onto the top of the specimen, so that the load was applied directly to the concrete. The tests were carried out at a loading rate of 0.3 mm per minute and were terminated until the concrete core touched the lower platen of the testing machine.

Fig. 3.12 gives the load–slippage relationships for the three specimens. Since there was no ultimate load and the load–slippage curve can be characterised as bi-linear, the critical load at which the two linear branches intersect was defined to be the failure load. During the tests, a slippage noise was heard at a load level of approximately 40% of the failure load. As shown in Fig. 3.12, there was a jump of slippage and drop of the applied load at this load level.

After the critical load was obtained, the bond strength was determined as the critical load divided by the area of the interface between the steel and the concrete. The bond strengths of Specimens A9-1, A9-2 and A10 were calculated as 0.68, 0.65 and 0.62 MPa, respectively. It was noted that Specimens A9-1 and A9-2 had the same section ($220 \times 170 \times 5.8$ mm) but different grades of concrete. Specimen A9-1 was in-filled with concrete whose maximum size of coarse aggregate was 20 mm, whilst the maximum size of coarse aggregate of Specimen A9-2 was 10 mm. The test results revealed that the proportion details of the concrete mix, especially the size of coarse aggregate can affect the bond strength of the interface between the steel section and the concrete. Moreover, the adverse effect of cross-sectional aspect ratio on the bond strength can be concluded by comparing test data of Specimens A9-2 and A10.

3.3 Experimental programme at HIT

A total of 22 columns were constructed and tested under concentric loading at HIT. The test variables were material strengths ($f_y=550$ MPa, $f'_c=61$ and 72 MPa), cross-sectional aspect ratio (1.0~2.0) and volumetric steel-to-concrete ratio (0.10~0.20). The failure loads, load–strain responses and load–shortening relationships of the columns were recorded for analysis.

3.3.1 Material properties

Welded steel hollow sections were used for constructing the composite columns, as shown in Fig. 3.13. All the component flat plates of the steel hollow sections were cut from a 1600-mm-wide steel coil in the longitudinal direction. The fabrication of the steel coil conformed to the requirements in ASTM (1997) for Steel A572 Grade 65. The specified minimum yield stress of the steel was 460 MPa. Six coupons were cut from the steel coil in the longitudinal direction and tested in tension to determine the actual material properties. Three of the coupons were prepared and tested in accordance with ASTM (1997), whilst the other three complied with the specifications in the Chinese testing standard GB2975 (1982). The dimensions of the coupons prepared according to GB2975 are shown in Fig. 3.2 (b). All the steel coupons were delivered to the Structural Mechanics Laboratory of NTU and tested in an Instron testing machine, as shown in Fig. 3.3(a). The loading rate was set at 0.5 mm per minute before the yield load and was increased to 1.5 mm per minute after the yield load to expedite the tests.

The detailed results of the tension tests on the steel coupons are summarised in Table 3.4. As the differences of yield stress and modulus of elasticity between the two standards were less than 2%, the material properties obtained according to GB2975 (1982) were used in the study. A typical stress–strain curve obtained from the coupons tests is illustrated in Fig. 3.4(b).

The concrete used to fill the steel hollow sections was mixed in two batches. From each batch, three concrete cubes (150×150×150 mm) were prepared. The concrete cubes were stripped from the moulds 24 hours after casting. The concrete cubes were exposed to the air in the laboratory with water sprayed once a day to maintain the humidity level. Concrete cylinders were not prepared due to the lack of

cylindrical moulds in the laboratory. At the same day as the CFSHS specimens were tested, the concrete cubes were crushed in a compression testing machine at a loading rate of approximately 330 kN per minute. The average compressive strengths of the two batches of concrete were determined as 71 and 82 MPa. The equivalent cylinder strengths were determined as 61 and 72 MPa, respectively based on the tables provided by Chen et al. (1996).

3.3.2 Fabrication of specimens

The fabrication process of welding four component plates to form a steel hollow section is shown in Fig. 3.14. After the fabrication of the steel hollow section was completed, the two ends were ground flat in a lathe, as illustrated in Fig. 3.15. This was because the four component plates were usually not flush at the ends after welding. A 10-mm-thick cap plate was welded to the base to hold the wet concrete. Concrete was then vertically poured into the steel hollow section in layers. Each layer of concrete was compacted using a poker vibrator. A 10-mm gap was left from the concrete surface to the top of the steel hollow section. Then the specimens were cured in the laboratory for 14 days with top open to the air for the concrete to set. Fig. 3.16 gives a picture of specimens in curing. As the concrete shrank over time, high-strength cement mortar was in-filled so that the concrete core was flush with the steel hollow section. Finally, a 10-mm-thick cap plate was welded to the top to form a complete specimen. The two cap plates were useful to ensure that the steel hollow section and the concrete core were simultaneously loaded during the test. Besides, local yielding of the steel section at the two ends can be avoided. The geometric and material properties of the 22 specimens are summarised in Table 3.5.

3.3.3 Test procedure and results

The specimens were tested in a 5000-kN-capacity universal testing machine in the Structures Laboratory of HIT. As illustrated in Fig. 3.17, the specimen was centrally positioned in the testing machine with four displacement transducers symmetrically mounted around it to measure the axial shortening during the test. Eight strain gauges were bonded at mid-height with two on each face of the steel section to record the longitudinal and transverse strains.

Prior to the actual test, a load level of 20 kN was applied so that the plattens of the testing machine were firmly attached to both ends of the specimen. Then the axial load was slowly applied by careful manipulation of the loading and unloading valves. The longitudinal and transverse strains as well as the axial shortening of the specimen were recorded at a load increment of 100 kN. It should be noted that the testing machine was load-control, so it was very difficult to ensure a low loading rate in the post-peak region. As shown in Fig. 3.18, the applied load decreased after the critical load, so it was hard to control the loading rate by manipulating the loading and unloading oil valves.

Three failure modes were identified for the specimens during the tests. For Specimens C4, C10-1, C11-1, C11-2, C12-1, C12-2, the concrete crushed abruptly in the post-peak region. As a result, only a short post-peak branch of the axial load–shortening curve was obtained for these six specimens. Two facts were considered to have contributed to the concrete crushing. One reason was the relative low volumetric steel-to-concrete ratios of these specimens. As a result, the steel section provided relatively little confinement to the high-strength concrete for improving its ductility performance. The other reason was the method of applying the load. Since the load was applied in a load-control method, it was hard to ensure a low loading rate when the applied load began to decrease. The energy exerted by the testing machine to the specimen could not be fully absorbed by the plastic deformation of the steel section and the concrete. As a result, crushing occurred to dissipate the excessive energy. Considering the favourable ductility performance of the specimens ($f_y=300$ MPa, $f'_c=83$ and 106 MPa) tested at NTU, a conclusion can be drawn that the use of displacement-control mode is important for an accurate measurement of the post-peak behaviour of high-strength rectangular CFSHS columns. The method of applying the load, whether it is displacement-control or load-control, can significantly affect the failure mode of the specimens. Displacement-control method is recommended for testing high-strength rectangular CFSHS columns.

Fracture of longitudinal welds was observed as the second type of failure mode. For Specimens C8-1 and C10-2, a complete collapse was noted shortly after the critical load. For the other 13 specimens, the failure was associated with the

outward local buckling of the steel section. These specimens demonstrated favourable post-yield behaviour, as shown in Fig. 3.18. A remarkable observation was that specimens with various cross-sectional aspect ratios showed different buckling modes. For specimens with $H/B=1.0$, local buckling occurred almost simultaneously and to the same extent on each face of the steel hollow sections. On the other hand, for specimens with $H/B=1.5$ and 2.0 , local buckling was more extensive and occurred earlier on the broader faces of the steel sections. The failure modes of the 22 specimens are illustrated in Fig. 3.19. This observation was identical to that of the specimens tested at NTU, as described in Section 3.2.

3.4 Experimental programme at FU

A total of 34 specimens were constructed and tested under concentric loading at FU. The primary test parameters were material strengths ($f_y=495$ MPa, $f'_c=55\sim 89$ MPa) and cross-sectional aspect ratio ($1.0\sim 2.0$). Material properties of steel and concrete were determined from respective tension and crushing tests. The load–strain and load–shortening relationships of the specimens were recorded for analysis.

3.4.1 Material properties

The hollow sections were fabricated by welding the four component flat plates in the same way as for specimens tested at NTU, as shown in Fig. 3.1 (b). All the flat plates were cut in the longitudinal direction from a 1400-mm-wide steel coil. The nominal yield stress of the steel coil was 450 MPa. In order to determine the actual material properties, four steel coupons were cut in the longitudinal direction from the steel coil and machined according to the requirements in ASTM (1997). The coupons were delivered to the Structural Mechanics Laboratory of NTU and tested in an Instron testing machine. The loading rate was 0.5 mm per minute before the yield load, whilst after the yield load it was 2.0 mm per minute. An extensometer with gauge length of 50 mm was clipped to the centre of the steel coupons to monitor the elongation. The average yield stress, tensile strength and modulus of elasticity were determined as 495, 581 and 206,000 MPa, respectively. Table 3.6 gives the results of the tension tests on the steel coupons. A typical stress–strain curve of the steel coupons is shown in Fig. 3.4 (c).

The concrete used to fill the steel hollow sections was mixed in three batches. From each batch, eight concrete cubes and four concrete cylinders were prepared. In order to simulate the curing condition of the concrete inside the steel hollow section, the concrete cylinders were cured in the moulds with top open to the air. Water was sprayed to the concrete cylinders twice a day to compensate for the moisture lost due to evaporation. The concrete cubes were removed from the moulds two days after casting and were then cured either in the fog room or exposed to the air in the laboratory, which will be described in the following subsection. On the same day as the CFSHS columns were tested, the concrete cylinders were removed from the moulds. The two ends of the concrete cylinders were ground flat using a disk grinder. The concrete cubes and cylinders were crushed at loading rates of approximately 330 and 265 kN/min, respectively. The details of mix proportions and measured material properties of concrete are summarised in Table 3.7.

3.4.2 Effect of curing condition

It is well understood that the development of concrete strength is influenced by the curing condition, because humidity and temperature can significantly affect the process of hydration. For concrete cured in steel hollow sections of composite columns, the rate of evaporation is expected to be very slow. This should be considered when curing concrete samples. In many research projects, such as Elremaily and Azizinamini (2002), the concrete samples were cured in the laboratory, exposed to the air until they were tested. This cast doubts if these concrete samples provided accurate estimates of strength for the concrete in composite columns. It should be noted that the standards for curing and testing concrete samples, such as ASTM (1997) requires that the concrete samples be cured in the fog room with constant temperature and humidity.

A test programme was carried out in the current study to investigate the effect of curing condition on the strength development of concrete. Eight concrete cubes in two series were cast and cured in the moulds for two days. One series of cubes were subsequently removed from the moulds and exposed to the air in the laboratory. Water was sprayed to the concrete samples twice a day to compensate for the moisture loss due to evaporation. The temperature in the laboratory during the period of curing the concrete samples varied from 18 to 26⁰C, whilst the relative

humidity was between 40 and 90%. On the other hand, the other series of concrete cubes were removed from the moulds and cured in the fog room. The temperature and relative humidity in the fog room were 20⁰C and 95%, respectively.

At the same day as the composite columns were tested, the concrete cubes were crushed in a 3000-kN-capacity testing machine. As shown in Table 3.8, the differences of the strengths of the two series of concrete cubes were as small as less than 0.3%. For concrete of Batch A, the concrete cubes cured in the fog room demonstrated an average strength (70.5 MPa) slightly higher than those cured in the laboratory (70.3 MPa). The difference was 0.28%. On the other hand, for concrete of Batch B, the concrete cubes cured in the fog room showed an average strength (72.7 MPa) lower than those cured in the laboratory (72.9 MPa). The difference was 0.27%. The comparison of results suggested that the variation of strengths for concrete samples cured in the fog room and exposed to the air in the laboratory was minimal. However, it should be noted that in the current study, the temperature and humidity conditions of the laboratory did not differ much from those in the fog room. It was recommended that concrete samples should be cured in the moulds until tested to simulate the real curing condition and to get a close estimate of strength for the concrete in composite columns.

3.4.3 Fabrication of specimens

After the welding process of the steel hollow section, both ends were ground flat. An 8-mm-thick cap plate was welded to its base to support the wet concrete. The concrete was vertically cast into the steel hollow section in layers. Each layer was compacted by a poker vibrator. Unlike the test programmes conducted at NTU and HIT, the concrete was cast about 5 mm higher than the steel section. Seven days later, the top of the concrete core was ground flat using a disk grinder so that it was flush with the steel section. Very little shrinkage and creep were observed before the top cap plate was welded 21 days after the casting of concrete. The material and geometric properties of the specimens are summarised in Table 3.9.

3.4.4 Test procedure and results

The specimens were tested 35 days after the casting of concrete. The axial load was applied by a 5000-kN-capacity universal testing machine in the Structures

Laboratory at FU. As shown in Fig. 3.20, the column was centrally positioned in the testing machine with four displacement transducers symmetrically placed around the column to measure the axial shortening. Four strain gauges were bonded on the exterior surfaces of the steel hollow section to monitor the longitudinal strains.

Prior to the actual test, a load level of 25 kN was applied so that the platens of the testing machine firmly contacted both ends of the column. The longitudinal strain and the axial shortening were recorded at a load increment of 100 kN. Fig. 3.21 illustrates axial load–shortening relationships obtained in the tests. The failure loads of the specimens are summarised in Table 3.9.

It was observed in the tests that the specimens with $f'_c = 55$ and 60 MPa exhibited favourable ductility performance. The applied axial load decreased slowly with large deformations after the peak load. On the other hand, most of the specimens with $f'_c = 89$ MPa experienced concrete crushing shortly after the peak load. A smooth and long post-peak response was not obtained for these columns. The reason was considered to be the brittle behaviour of the concrete with a compressive strength as high as 89 MPa. However, it was noted that Specimens R13-1 and R13-2 with $f'_c = 89$ MPa did not experience concrete crushing due to the relative high volumetric steel-to-concrete ratio. The smooth and stable post-peak behaviour of these two specimens, as shown in Fig. 3.21 suggests that high-strength concrete can exhibit favourable ductility performance if the volumetric steel-to-concrete ratio is high enough.

It is worth noting that the composite columns with $f'_c = 106$ MPa tested at NTU exhibited favourable post-peak behaviour without crushing of concrete. Those specimens were tested in a displacement-control mode. The different ductility performances of specimens tested at NTU and FU suggest that high-strength composite columns should be tested in a displacement-control mode if the post-peak response is to be examined.

Two types of failure modes were identified for the specimens in the tests. The first type of failure was abrupt concrete crushing for specimens with $f'_c = 89$ MPa except Specimens R13-1 and R13-2. These specimens experienced a sharp drop of the applied load due to the crushing of concrete. The other type of failure occurred

for all the specimens with $f'_c = 55$ and 60 MPa. It was characterised as outward local buckling of the steel section. It was noted that the buckling mode was affected by the cross-sectional aspect ratio. As shown in Fig. 3.22, local buckling occurred almost to the same extent on each face for specimens with $H/B=1.0$, whilst for specimens with $H/B=1.3\sim 2.0$, local buckling on the two broader faces was more extensive and occurred earlier than the other two faces.

In order to investigate the failure mode of the concrete, the steel sections of Specimens R1-1, R3-1, R5-2, R7-2, R9-2 and R11-2 were flame cut after the tests. As shown in Fig. 3.23, for Specimens R1-1, R3-1 and R5-2 which experienced large deformation, the concrete crushed upon the termination of the tests. For the other three specimens, slight concrete crushing was observed at the position where the steel section locally buckled.

One of the objectives of the current test programme is to investigate the effect of cross-sectional aspect ratio on the strength increase of the concrete. Since the rectangular steel hollow section confines the concrete core by plate bending, the strength increase resulting from confinement is likely to decrease with the increase of the H/B ratio. Current tests demonstrate such an adverse effect of the H/B ratio on the strength increase of the concrete. The relationships of H/B ratio and strength index (SI) for the specimens are given in Fig. 3.24 for the six series of specimens tested in the study. SI is defined as in Equation (3.7).

$$SI = \frac{N_e - A_s f_y}{0.85 A_c f'_c} \quad (3.7)$$

It is noted that the specimens in each series have identical material strengths and volumetric steel-to-concrete ratio, but their cross-sectional aspect ratios are different. Results show that specimens with higher H/B ratios tend to have lower strength increase of the concrete. For columns with $H/B = 2.0$, the strength improvement is minimal. Therefore, high-strength rectangular CFSHS columns with $H/B > 2.0$ are not recommended for design.

3.5 Concluding remarks

Three experimental programmes on the behaviour of high-strength rectangular CFSHS stub columns under concentric loading have been described. The experimental results of 73 columns tested in the study are given. It is noted that the strength enhancement of the confined concrete is affected by the volumetric steel-to-concrete ratio, material properties of steel and concrete and cross-sectional aspect ratio. In order to effectively employ rectangular steel hollow sections to improve the strength of composite columns, it is recommended that CFSHS columns with H/B ratios higher than 2.0 should not be used.

The present tests also show the ductility enhancement of high-strength concrete confined by steel hollow sections. Most of the composite columns with high-strength concrete exhibited large deformations in the tests. Compared with the abrupt crushing at the peak load for un-confined high-strength concrete, it can be commented that the ductility performance of high-strength concrete in CFSHS columns is significantly improved due to confinement effect. However, it is noted that the method of applying the load can affect the post-peak behaviour of the column. Displacement-control method is recommended for investigating the axial load behaviour of high-strength rectangular CFSHS columns.

Push-out tests on three specimens carried out in the study are also presented. The test results suggest that the mix proportion details of the concrete, especially the size of coarse aggregate can affect the bond strength of the interface between the steel and the concrete. Moreover, bond strength is adversely affected by the cross-sectional aspect ratio. Rectangular CFSHS columns with higher H/B ratios tend to have lower bond strength.

A research programme aimed at investigating the effect of curing condition on the strength development of concrete has been described. Results indicate little difference of strength between the concrete samples cured in the fog room and those cured in the laboratory. However, in order to simulate the real curing condition and to get a close estimate of strength for the concrete in composite columns, it is recommended that concrete samples be cured in the moulds until they are tested.

Table 3.1 Material properties of steel (NTU)

Coupon no.	f_y (MPa)	f_u (MPa)	f_y/f_u	E_s (GPa)
1	296	473	0.63	204
2	300	474	0.64	203
3	305	476	0.65	201
Average	300	474	0.64	203

Table 3.2 Details of high-strength concrete (NTU)

Batch no.	A	B
Cement (Ordinary portland cement) (kg/m ³)	480	550
Silica fume (kg/m ³)	20	40
Water (kg/m ³)	170	165
Maximum aggregate size (mm)	20	10
Coarse aggregate (kg/m ³)	1025	950
Sand (kg/m ³)	665	661
Water-reducing admixture (ml/m ³)	5760	4720
Superplasticizing admixture (ml/m ³)	0	5900
Modulus of elasticity (GPa)	41	43
Average cube strength (MPa)	93	115
Average cylinder strength (MPa)	83	106

Table 3.3 Summary of specimens tested at NTU

No.	$H \times B \times t$ (mm)	L (mm)	H/t	H/B	f'_c (MPa)	f_y (MPa)	α	N_e (kN)	N_{buc} (kN)
A1	120×120×5.8	360	21	1.0	83	300	0.23	1697	1300
A2	120×120×5.8	360	21	1.0	106	300	0.23	1919	1220
A3-1	200×200×5.8	600	34	1.0	83	300	0.13	3996	2700
A3-2	200×200×5.8	600	34	1.0	83	300	0.13	3832	2300
A4-1	130×100×5.8	390	22	1.3	83	300	0.24	1601	1260
A4-2	130×100×5.8	390	22	1.3	83	300	0.24	1566	1200
A5-1	130×100×5.8	390	22	1.3	106	300	0.24	1854	1140
A5-2	130×100×5.8	390	22	1.3	106	300	0.24	1779	1100
A6-1	220×170×5.8	660	38	1.3	83	300	0.13	3684	2200
A6-2	220×170×5.8	660	38	1.3	83	300	0.13	3717	2040
A7-1	180×100×5.8	540	31	1.8	83	300	0.21	2059	1400
A7-2	180×100×5.8	540	31	1.8	83	300	0.21	2019	1330
A8-1	180×100×5.8	540	31	1.8	106	300	0.21	2287	1300
A8-2	180×100×5.8	540	31	1.8	106	300	0.21	2291	1300
A9-1*	220×170×5.8	660	38	1.3	106	300	0.13	300	NA
A9-2*	220×170×5.8	660	38	1.3	106	300	0.13	287	NA
A10*	220×150×5.8	660	47	1.8	83	300	0.13	346	NA

*Note: Specimens A9-1, A9-2 and A10 were for push-out tests.

Table 3.4 Material properties of steel (HIT)

Testing standard	Coupon No.	f_y (MPa)	f_u (MPa)	f_y/f_u	E_s (GPa)
GB2975	1	532	582	0.914	206
	2	558	623	0.896	206
	3	559	619	0.903	210
	Average	550	608	0.905	207
ASTM	4	519	601	0.864	202
	5	555	612	0.907	203
	6	551	621	0.887	203
	Average	542	611	0.887	203

Table 3.5 Summary of specimens tested at HIT

No.	$H \times B \times t$ (mm)	L (mm)	H/t	H/B	f'_c (MPa)	f_y (MPa)	α	N_e (kN)
C1-1	100×100×4.2	300	24	1.0	61	550	0.19	1490
C1-2	100×100×4.2	300	24	1.0	61	550	0.19	1535
C2-1	100×100×4.2	300	24	1.0	72	550	0.19	1740
C2-2	100×100×4.2	300	24	1.0	72	550	0.19	1775
C3	180×180×4.2	540	43	1.0	61	550	0.10	3590
C4	180×180×4.2	540	43	1.0	72	550	0.10	4210
C5-1	120×80×4.2	360	29	1.5	61	550	0.20	1450
C5-2	120×80×4.2	360	29	1.5	61	550	0.20	1425
C6-1	120×80×4.2	360	29	1.5	72	550	0.20	1560
C6-2	120×80×4.2	360	29	1.5	72	550	0.20	1700
C7-1	180×120×4.2	540	43	1.5	61	550	0.13	2530
C7-2*	180×120×4.2	540	43	1.5	61	550	0.13	NA
C8-1	180×120×4.2	540	43	1.5	72	550	0.13	2970
C8-2	180×100×4.2	540	43	1.5	72	550	0.13	2590
C9-1	160×80×4.2	480	38	2.0	61	550	0.18	1710
C9-2	160×80×4.2	480	38	2.0	61	550	0.18	1820
C10-1	160×80×4.2	480	38	2.0	72	550	0.18	1880
C10-2	160×80×4.2	480	38	2.0	72	550	0.18	2100
C11-1	200×100×4.2	600	48	2.0	61	550	0.14	2350
C11-2	200×100×4.2	600	48	2.0	61	550	0.14	2380
C12-1	200×100×4.2	600	48	2.0	72	550	0.14	2900
C12-2	200×100×4.2	600	48	2.0	72	550	0.14	2800

* The test data of Specimen C7-2 were not recorded.

Table 3.6 Material properties of steel (FU)

Coupon no.	f_y (MPa)	f_u (MPa)	f_y/f_u	E_s (GPa)
1	498	583	0.85	209
2	495	582	0.85	205
3	492	579	0.85	204
Average	495	581	0.85	206

Table 3.7 Details of high-strength concrete (FU)

Batch no.	A	B	C
Ordinary portland cement (kg/m ³)	480	550	520
Silica fume (kg/m ³)	0	0	90
Fly ash (kg/ m ³)	30	70	0
Water (kg/m ³)	160	158	140
Maximum aggregate size (mm)	10	10	10
Coarse aggregate (kg/m ³)	1120	1170	1150
Sand (kg/m ³)	620	530	550
Water-reducing admixture (FDN) (kg/m ³)	10.2	12.4	12.2
Modulus of elasticity (GPa)	38	39	42
Average cube strength (MPa)	70	73	98
Average cylinder strength (MPa)	55	60	89

Table 3.8 Effect of curing condition on strength of concrete cubes

Concrete batch no.	Curing condition	Cube no.	Cube strength (MPa)	Average strength (MPa)
A	Exposed to the air	1	65.6	70.3
		2	73.9	
		3	69.4	
		4	72.4	
	Cured in fog room	1	67.7	70.5
		2	69.7	
		3	71.3	
		4	73.2	
B	Exposed to the air	1	74.7	72.9
		2	71.5	
		3	74.4	
		4	71.1	
	Cured in fog room	1	73.3	72.7
		2	72.4	
		3	73.9	
		4	71.2	

Table 3.9 Summary of specimens tested at FU

No.	$H \times B \times t$ (mm)	L (mm)	H/B	H/t	f'_c (MPa)	f_y (MPa)	α	N_e (kN)
R1-1	120×120×4	360	1.0	30	55	495	0.15	1739
R1-2	120×120×4	360	1.0	30	55	495	0.15	1718
R2-1	120×120×4	360	1.0	30	60	495	0.15	1701
R2-2	120×120×4	360	1.0	30	60	495	0.15	1657
R3-1	150×100×4	450	1.5	38	55	495	0.15	1815
R3-2	150×100×4	450	1.5	38	55	495	0.15	1763
R4-1	150×100×4	450	1.5	38	60	495	0.15	1735
R4-2	150×100×4	450	1.5	38	60	495	0.15	1778
R5-1	180×90×4	540	2.0	45	55	495	0.15	1725
R5-2	180×90×4	540	2.0	45	55	495	0.15	1742
R6-1	180×90×4	540	2.0	45	60	495	0.15	1773
R6-2	180×90×4	540	2.0	45	60	495	0.15	1795
R7-1	130×130×4	390	1.0	33	55	495	0.14	1963
R7-2	130×130×4	390	1.0	33	55	495	0.14	1988
R8-1	130×130×4	390	1.0	33	60	495	0.14	2020
R8-2	130×130×4	390	1.0	33	60	495	0.14	2018
R9-1	160×110×4	480	1.5	40	55	495	0.14	1947
R9-2	160×110×4	480	1.5	40	55	495	0.14	1912
R10-1	160×110×4	480	1.5	40	60	495	0.14	1982
R10-2	160×110×4	480	1.5	40	60	495	0.14	1923
R11-1	190×100×4	570	1.9	48	55	495	0.14	2035
R11-2	190×100×4	570	1.9	48	55	495	0.14	2138
R12-1	190×100×4	570	1.9	48	60	495	0.14	2049
R12-2	190×100×4	570	1.9	48	60	495	0.14	2124
R13-1	106×106×4	320	1.0	27	89	495	0.17	1749
R13-2	106×106×4	320	1.0	27	89	495	0.17	1824
R14-1	130×90×4	390	1.4	33	89	495	0.17	1752
R14-2	130×90×4	390	1.4	33	89	495	0.17	1806
R15-1	160×80×4	480	2.0	40	89	495	0.17	1878
R15-2	160×80×4	480	2.0	40	89	495	0.17	1858
R16-1	140×140×4	420	1.0	35	89	495	0.13	2752
R16-2	140×140×4	420	1.0	35	89	495	0.13	2828
R17-1	160×125×4	480	1.3	40	89	495	0.13	2580
R17-2	160×125×4	480	1.3	40	89	495	0.13	2674

(a) Elevation view

(b) Cross-sectional view

Fig. 3.1 CFSHS specimens tested at NTU

(a) ASTM

(b) GB 2975

Fig. 3.2 Dimensions of steel coupons

(a) Coupon in test

(b) Coupons after tests

Fig. 3.3 Test setup for steel coupons

(a) Specimens tested at NTU

(b) Specimens tested at HIT

(c) Specimens tested at FU

Fig. 3.4 Stress-strain relationships of steel coupons

(a) Concrete cube during test

(b) Concrete cube after test

Fig. 3.5 Test setup for concrete cubes

(a) Concrete cylinder before test

(b) Concrete cylinder during test

(d) Concrete cylinder after test

Fig. 3.6 Test setup for concrete cylinders

(a) Steel hollow sections

(b) Specimens in curing

Fig. 3.7 Fabrication of CFSHS specimens at NTU

(a) Specimen in the test rig

(b) Strain gauge locations

Fig. 3.8 Test setup for concentric loading tests at NTU

Fig. 3.9 Load-shortening curves for specimens tested at NTU

Fig. 3.9 Load-shortening curves for specimens tested at NTU (cont'd)

Fig. 3.9 Load-shortening curves for specimens tested at NTU (cont'd)

(a) Fracture of weld

(b) Buckling of steel sections

Fig. 3.10 Failure modes of specimens tested at NTU

Fig. 3.11 Schematic view of test setup for push-out tests

Fig. 3.12 Load–slippage relationships for specimens

Fig. 3.13 Cross-sectional view of CFSHS column (HIT)

(a) Component plates of steel hollow sections

(b) Bracing for welding

(c) Tack welding

(d) Welding completed

Fig. 3.14 Process of welding steel hollow sections

Fig. 3.15 Steel hollow section in lathe

Fig. 3.16 Specimens in curing at HIT

(a) Specimen in the test rig

(b) Strain gauge locations

Fig. 3.17 Test setup for concentric loading tests at HIT

Fig. 3.18 Load-shortening curves for specimens tested at HIT

(d)

(e)

(f)

Fig. 3.18 Load-shortening curves for specimens tested at HIT (cont'd)

Fig. 3.18 Load-shortening curves for specimens tested at HIT (cont'd)

(j)

(k)

Fig. 3.18 Load-shortening curves for specimens tested at HIT (cont'd)

(a) Fracture of weld

(b) Buckling of specimens

Fig. 3.19 Failure modes of specimens tested at HIT

(a) Specimen in the test rig

(b) Strain gauge locations

Fig. 3.20 Test setup for concentric loading tests at FU

Fig. 3.21 Load-shortening curves for specimens tested at FU

Fig. 3.21 Load-shortening curves for specimens tested at FU (cont'd)

Fig. 3.21 Load-shortening curves for specimens tested at FU (cont'd)

(j)

(k)

(l)

Fig. 3.21 Load-shortening curves for specimens tested at FU (cont'd)

(m)

(n)

(o)

Fig. 3.21 Load-shortening curves for specimens tested at FU (cont'd)

(p)

(q)

Fig. 3.21 Load-shortening curves for specimens tested at FU (cont'd)

Fig. 3.22 Failure modes of specimens tested at FU

Fig. 3.23 Failure modes of concrete for specimens tested at FU

Fig. 3.24 Effect of H/B ratio on strength increase of concrete

CHAPTER 4

ANALYTICAL STUDY OF RECTANGULAR CFSHS COLUMNS UNDER CONCENTRIC LOADING

4.1 Introduction

This chapter presents an analytical study of the axial load behaviour of high-strength rectangular CFSHS stub columns. A numerical model is proposed to predict the behaviour of rectangular CFSHS columns subjected to concentric loading. The development of the model encompasses proposing respective constitutive relationships for steel and concrete and constructing a computer programme for analysis. The proposed analytical model is calibrated against the experimental results of the 70 columns described in Chapter 3 and those reported by other researchers. Both the ultimate capacity and the load–strain response of the columns are compared between the tests and the analytical model to assess the suitability of the model for predicting the axial load behaviour of high-strength rectangular CFSHS stub columns.

Section 4.2 describes the effect of local buckling of steel on the behaviour of CFSHS columns. The requirements in the design codes for ensuring a compact cross-section are reviewed. An effective width method is presented to consider the local buckling effect in the calculation of axial load capacity of CFSHS stub columns. Section 4.3 reviews the design specifications in EC4, ACI and AISC regarding the determination of ultimate capacity of CFSHS stub columns. Section 4.4 presents the proposed analytical model. In this section, the material constitutive relationships and the assumptions made in the development of the analytical model are discussed in detail. Section 4.5 provides the calibration of the analytical model with the test results. Both the ultimate capacity and the axial load–strain response between the tests and the model are commented. Section 4.6 gives the conclusions of this analytical study.

4.2 Local buckling

The steel hollow section of composite columns cannot fully develop its material strength if premature local buckling occurs. Although the in-filled concrete prevents the steel hollow section from buckling inward, the outward buckling remains unaffected. In the current study, the risk of premature local buckling is checked using the allowable H/t limits specified in the design codes. If the plate slenderness exceeds the allowable H/t limits, an effective width method developed by Bradford (1995) is employed to consider the local buckling effect.

The elastic buckling stress of a rectangular plate under uniform compression can be calculated as (Uy and Bradford 1996)

$$\sigma_b = \frac{\gamma \pi^2 E_s}{12(1 - \mu^2)(H/t)^2} \quad (4.1)$$

in which, μ and γ are the Poisson's ratio and buckling coefficient, respectively. $\gamma=10.31$ was suggested by Uy and Bradford (1996) for steel plates in contact with a rigid medium, such as the in-filled concrete.

For steel plates without restraint from medium such as in-filled concrete, the buckling coefficient (γ) can be determined based on the length-to-width ratio of the steel plate and the boundary conditions along the longitudinal edges. For plates with large length-to-width ratio, γ only depends on the boundary conditions of the longitudinal edges. The research findings by Varma (2000) showed that γ can be taken as 4.0 for simply supported plates and 6.97 for fixed supported plates. If σ_b is taken as f_y in Equation (4.1), the following H/t limits can be obtained.

$$H/t = \sqrt{3.61 E_s / f_y} \quad \text{for longitudinally simply supported plates} \quad (4.2a)$$

$$H/t = \sqrt{6.29 E_s / f_y} \quad \text{for longitudinally fixed supported plates} \quad (4.2b)$$

Different allowable H/t limits are provided in the design codes such as EC4, ACI and AISC to ensure a compact composite section. EC4 specifies the allowable H/t limit of a rectangular CFSHS column as

$$H/t \leq 52\sqrt{235/f_y} = \sqrt{3.18E_s/f_y} \quad (4.3)$$

On the other hand, both AISC and ACI require that

$$H/t \leq \sqrt{3E_s/f_y} \quad (4.4)$$

The following conclusions can be made by comparing Equations (4.2) to (4.4): (a) the three codes assume the component plates of a rectangular CFSHS column to be longitudinally simply supported by the adjacent component plates. The restraint from the adjacent plates is omitted; (b) the stability enhancement of the steel section due to the in-filled concrete is not considered and (c) the effect of residual stresses due to welding on the stability performance of steel plates is neglected.

In order to incorporate the local buckling effect in determining the axial load capacity of composite columns with large H/t ratios, Bradford (1995) reported an effective width approach. The cross-sectional area of the steel plates with local buckling is reduced in the calculation. The effective width (H_e) is determined as

$$H_e/H = \alpha_r \sqrt{\sigma_b/f_y} \quad (4.5)$$

in which, σ_b can be calculated from Equation (4.1), α_r is a parameter accounting for residual stresses and geometric imperfections. For steel plates supported on two longitudinal edges and subjected to pure compression, $\alpha_r = 0.65$ and $\alpha_r = 0.74$ were suggested by Bradford et al. (1987) for steel plates which were heavily welded and lightly welded, respectively.

Based on Equations (4.1) and (4.5), the following allowable limit can be obtained for ensuring a compact composite section (i.e. $H_e = H$).

$$H/t = \sqrt{9.31\alpha_r^2 E_s/f_y} \quad (4.6)$$

It should be noted that the experimental results reported by Uy (2001a) showed that the effective width approach proposed by Bradford (1995) was conservative for CFSHS columns. Extensive research is needed for proposing a reliable and accurate approach to consider the local buckling effect.

4.3 Code specifications

According to EC4, the ultimate capacity of an axially loaded CFSHS stub column can be determined by summing up the strengths of the steel section and the concrete core, as in Equation (4.7). The coefficient of 0.85, which applies to the strength of concrete in reinforced concrete columns, is taken as 1.0 for CFSHS columns to account for the strength increase of the concrete due to confinement effect. The use of EC4 is limited to composite columns with concrete cylinder strength and steel yield stress not higher than 50 and 355 MPa, respectively.

$$N_u = A_s f_y + A_c f_c' \quad (4.7)$$

In ACI, the axial load capacity of CFSHS columns is evaluated the same as that of reinforced concrete columns, as in Equation (4.8). A coefficient of 0.85 is included in the cylinder strength of concrete to account for long-term and size effects. The strength enhancement of the concrete due to confinement by the steel hollow section is not considered.

$$N_u = A_s f_y + 0.85 A_c f_c' \quad (4.8)$$

The AISC approach to predict the ultimate capacity of CFSHS columns is essentially identical to that of steel columns. According to the AISC specifications, the axial load capacity of CFSHS columns can be evaluated by converting the concrete strength into an equivalent steel strength, as in Equation (4.9). The AISC approach is applicable to the design of composite columns with concrete cylinder strength and steel yield stress lower than 55 and 415 MPa, respectively.

$$N_u = A_s f_{cr} \quad (4.9)$$

where,

$$f_{cr} = (0.658^{\lambda_c^2}) f_{my} \quad \text{for } \lambda_c \leq 1.5$$

$$f_{cr} = (0.877 / \lambda_c^2) f_{my} \quad \text{for } \lambda_c > 1.5$$

$$\lambda_c = \frac{L}{r_s \pi} \sqrt{f_{my} / (E_s + 0.4 E_c A_c / A_s)}$$

$$f_{my} = f_y + 0.85 f'_c (A_c / A_s)$$

$$r_s = \text{radius of gyration of steel hollow section}$$

4.4 Proposed analytical model

Fibre-based analytical models have been widely used for numerical analysis of composite columns by many researchers, such as Tomii et al. (1977), Susantha et al. (2001), Uy (2000, 2001a), Han (2004a), Sakino et al. (2004) and Fujimoto et al. (2004). These models are based on the strain compatibility and material constitutive relationships. The discrepancy between these models is the different constitutive relationships employed for the steel and the concrete. Strength increase of the concrete was considered in Tomii et al. (1977) and Han (2004a), but ignored in Uy (2001a) and Sakino et al. (2004).

In order to estimate the non-linear behaviour of CFSHS columns, a fibre-based analytical model is constructed in the current study based on the following assumptions: (a) there is full strain compatibility between the steel and the concrete and (b) only the longitudinal stress and strain are considered.

A four-branch material model is developed in the analytical study to simulate the stress-strain behaviour of the steel, as illustrated in Equation (4.10) and Fig. 4.1(a). This stress-strain model incorporates elastic, plastic and strain hardening behaviours of the steel material.

$$\sigma_s = E_s \varepsilon_s \quad \text{for } \varepsilon_s \leq \varepsilon_1 \quad (4.10a)$$

$$\sigma_s = f_y \quad \text{for } \varepsilon_1 < \varepsilon_s \leq \varepsilon_2 \quad (4.10b)$$

$$\sigma_s = f_u - (f_u - f_y)(\varepsilon_3 - \varepsilon_s)^2 / (\varepsilon_3 - \varepsilon_2)^2 \quad \text{for } \varepsilon_2 < \varepsilon_s \leq \varepsilon_3 \quad (4.10c)$$

$$\sigma_s = f_u \quad \text{for } \varepsilon_s \geq \varepsilon_3 \quad (4.10d)$$

in which, $\varepsilon_1 = f_y / E_s$, ε_2 is the strain at which strain hardening initiates, ε_3 is the strain corresponding to the tensile strength of steel. ε_2 and ε_3 can be determined from coupon tests. If they are not available, the following values are recommended for high-strength steel: $\varepsilon_2 = 10 \varepsilon_1$; $\varepsilon_3 = 50 \varepsilon_1$.

A material model as shown in Fig. 4.1(b) is constructed in the analytical study to simulate the stress–strain behaviour of confined concrete in rectangular CFSHS columns. Among the many factors which influence the stress–strain response of high-strength concrete confined by rectangular steel sections, the following three are considered in the current study: (a) cross-sectional aspect ratio; (b) confinement factor, which denotes the amount of confinement of the concrete by the steel hollow section and (c) compressive strength of unconfined concrete. The constitutive relationship of confined concrete is shown in Equation (4.11). ε_0 and f'_c in this equation are in $\mu\varepsilon$ and MPa, respectively. The coefficients and the sub-equations in Equation (4.11) are determined by comparing the predicted load–shortening curves with experimental results in wide ranges of test variables. A software DATAFIT is employed for determining the coefficients in the numerical analysis. It should be noted that Equation (4.11b) was modified from the previously proposed equation of Sargin (1971) to simulate the stress–strain response of concrete confined by rectangular steel hollow sections.

$$y = \frac{1.6x}{0.6 + x} \quad \text{for } x \leq 1 \quad (4.11a)$$

$$y = \frac{Ux + Vx^2}{1 + (U - 2)x + (V + 1)x^2} \quad \text{for } x > 1 \quad (4.11b)$$

in which,

$$y = \sigma_c / f_0$$

$$\sigma_c = \text{stress of concrete}$$

$$f_0 = 0.85 f'_c \eta$$

$$\eta = 1 + \frac{[1 - 0.65(\beta - 1)^3] (0.37 \xi^{0.82} - 0.19 \xi)}{1 + 0.05 (f'_c / 50)^{5.65}}$$

$$\xi = A_s f_y / (0.85 A_c f'_c), \text{ confinement factor}$$

$$\beta = H / B, \text{ cross-sectional aspect ratio } (B \leq H)$$

$$x = \varepsilon_c / \varepsilon_0$$

$$\varepsilon_c = \text{strain of concrete}$$

$$\varepsilon_0 = 1300 + (1300 + 18 f'_c) \xi^{0.2}$$

$$U = -0.09 - 0.06 \beta$$

$$V = (0.36 - 0.08 \beta) \xi^{0.3}$$

As shown in Figs. 4.2 and 4.3, this material model incorporates the adverse effect of H/B ratio and the positive effect of confinement factor on the stress–strain response of concrete confined by rectangular steel hollow sections.

4.5 Calibration of proposed model

4.5.1 Axial load–strain behaviour

The axial load–strain relationships obtained in the experimental programme described in Chapter 3 are employed to calibrate the proposed analytical model. The experimental load–strain responses of the 70 columns are compared with the curves predicted by the analytical model, as shown in Figs. 4.4~4.6.

In Fig. 4.4, the analytical model is compared with the test curves of the 14 specimens with $f_y = 300$ MPa and $f'_c = 83$ and 106 MPa tested at NTU. All the test curves have a smooth descending branch without abrupt crushing of concrete. Moreover, these specimens demonstrate a linear ascending branch of the load–strain curve, which is not very well simulated by the proposed model. However, the failure load, as well as the descending branch of the load–strain curve is accurately predicted by the model.

Fig. 4.5 illustrates the comparison of the experimental load–strain relationships obtained at HIT and the predicted curves using the proposed model. The 22 specimens were constructed from high-strength materials with $f_y = 550$ MPa and $f'_c = 61$ and 72 MPa. Results suggest that the analytical model agrees reasonably well with the experimental results for most specimens. However, for Specimens C4, C8,

C10, C11 and C12, abrupt concrete crushing occurred in the tests shortly after the peak load. Hence, the post-peak branch of the experimental load–strain response was not obtained. A close correlation of the analytical model with the tests in the post-peak region was not achieved for these specimens due to the lack of available test data.

Fig. 4.6 gives the experimental and analytical load–strain relationships for the 34 specimens tested at FU. These composite columns were constructed from high-strength materials ($f_y = 495$ MPa, $f'_c = 55 \sim 89$ MPa). Smooth post-peak curves were obtained for most specimens with $f'_c = 55$ and 60 MPa, which did not exhibit abrupt concrete crushing. For these columns, the analytical model closely predicts the non-linear behaviour compared with the experimental results. For the specimens with $f'_c = 89$ MPa, abrupt concrete crushing occurred except Specimens R13-1 and R13-2. The analytical model agrees very well with the experimental load–strain relationships for these columns before the crushing of concrete. It can be concluded that the proposed analytical model is capable of accurately predicting the axial load–strain relationships for the 34 specimens.

From the comparison of load–strain response between the tests and the proposed analytical model, it can be concluded that the proposed model is suitable for estimating the behaviour of high-strength rectangular CFSHS stub columns subjected to concentric loading.

4.5.2. *Ultimate capacity*

In this section, the failure loads of the 70 specimens obtained in the experiments are compared with the ultimate capacities calculated using the proposed model and the specifications in the design codes EC4, ACI and AISC. The comparison of the ultimate capacities is summarised in Tables 4.1~4.3. In these tables, N_e and N_c denote the experimental and calculated failure loads, respectively.

Table 4.1 gives the comparison of failure loads between the tests carried out at NTU, design codes and the proposed model. Results suggest that EC4 is unsafe to determine the axial load capacity of the 14 stub columns fabricated from mild steel sections ($f_y = 300$ MPa) and high-strength concrete ($f'_c = 83$ and 106 MPa). The average difference of the experimental and predicted ultimate capacities is 6%,

whilst the minimum and maximum N_e/N_c ratios are 0.90 and 0.99, respectively. ACI and AISC, on the other hand conservatively predict the failure loads by 3 and 4%, respectively. The proposed model is more accurate than the design codes with an average N_e/N_c ratio of 1.00 and a standard deviation of 0.03. The minimum and maximum N_e/N_c ratios are 0.96 and 1.07, respectively.

Table 4.2 illustrates the experimental results of the 21 stub columns tested at HIT and the calculated failure loads using the design codes and the proposed model. Comparison indicates that EC, ACI and AISC are conservative to predict the axial load capacities by 7, 14 and 16%, respectively. The proposed model, on the other hand underestimates the failure loads by 9% with a coefficient of variation of 0.06. The minimum and maximum N_e/N_c ratios are 0.97 and 1.19, respectively.

Table 4.3 shows the calibration of the design codes and the proposed model against the test data of the 34 stub columns tested at FU. The comparison of results suggests that EC4 gives the most accurate estimate of the failure loads with an average difference of 2%. The minimum and maximum N_e/N_c ratios are 0.94 and 1.10, respectively. ACI and AISC on the other hand underestimate the axial load capacities by 10 and 12%, respectively. The proposed model also conservatively predicts the failure loads with an average difference of 4% and a coefficient of variation of 0.04. The minimum and maximum N_e/N_c ratios are 0.98 and 1.14, respectively.

Fig. 4.7 illustrates the calibration of the proposed model against the test data obtained in the three experimental programmes. It can be concluded that the model gives a close and conservative estimate of the axial load capacity for most of the specimens.

The proposed analytical model is also verified with the test data pertaining to rectangular CFSHS stub columns reported by other researchers, i.e., Shiiba and Harada (1994), Schneider (1998), Varma et al. (2000), Uy (2001a), Han (2002) and Sakino et al. (2004). The test variables in these research projects are in wide ranges with $f_y = 228 \sim 825$ MPa, $f_c' = 23 \sim 110$ MPa, $H/B = 1.0 \sim 2.0$ and $\alpha = 0.06 \sim 0.26$. As illustrated in Fig. 4.8, the proposed analytical model agrees very well with most of the experimental data. Calibration of the proposed model suggests that it is capable

of predicting the axial load capacity for not only high-strength but also normal strength rectangular CFSHS columns.

4.6 Concluding remarks

An analytical study of the axial load behaviour of high-strength rectangular CFSHS columns has been described. A numerical model is constructed to estimate the behaviour of high-strength rectangular CFSHS columns subjected to concentric loading. Respective constitutive relationships are developed for the steel and the concrete. The calibration of the analytical model against the test results of the 70 columns described in Chapter 3 has been presented. Results reveal that the proposed analytical model can predict the non-linear behaviour of the high-strength rectangular CFSHS columns with reasonable accuracy. The test data reported in the literature on rectangular CFSHS columns subjected to concentric loading are also employed to calibrate the proposed analytical model. Comparison of ultimate capacities between the model and the tests shows that the model can be used for estimating the axial load capacity for not only high-strength but also normal strength rectangular CFSHS columns.

The design specifications in the codes (EC4, ACI and AISC) pertaining to CFSHS columns are also calibrated against the test data obtained in the three experimental programmes. Comparison suggests that the provisions in ACI and AISC are conservative to predict the axial load capacity of the specimens. However, EC4 yields un-conservative estimates of the failure loads for the composite columns fabricated from mild steel and high-strength concrete. Revisions to the three design codes are required so that they can be extended to the design of high-strength CFSHS columns subjected to concentric loading.

Table 4.1 Comparison of tests (NTU), codes and proposed model

No.	$H \times B \times t$ (mm)	f'_c (MPa)	f_y (MPa)	N_e (kN)	N_e / N_c			
					EC4	ACI	AISC	Proposed model
A1	120×120×5.8	83	300	1697	0.96	1.05	1.05	1.00
A2	120×120×5.8	106	300	1919	0.94	1.04	1.04	1.02
A3-1	200×200×5.8	83	300	3996	0.93	1.04	1.04	0.99
A3-2	200×200×5.8	83	300	3832	0.90	1.00	1.00	0.96
A4-1	130×100×5.8	83	300	1601	0.98	1.07	1.08	1.02
A4-2	130×100×5.8	83	300	1566	0.96	1.05	1.05	1.00
A5-1	130×100×5.8	106	300	1854	0.99	1.09	1.10	1.07
A5-2	130×100×5.8	106	300	1779	0.95	1.04	1.06	1.03
A6-1	220×170×5.8	83	300	3684	0.91	1.01	1.02	0.97
A6-2	220×170×5.8	83	300	3717	0.92	1.02	1.03	0.98
A7-1	180×100×5.8	83	300	2059	0.95	1.04	1.05	1.01
A7-2	180×100×5.8	83	300	2019	0.93	1.02	1.03	0.99
A8-1	180×100×5.8	106	300	2287	0.91	1.01	1.02	0.99
A8-2	180×100×5.8	106	300	2291	0.91	1.01	1.03	0.99
Average					0.94	1.03	1.04	1.00
SD					0.03	0.03	0.02	0.03
COV					0.03	0.03	0.02	0.03

Table 4.2 Comparison of tests (HIT), codes and proposed model

No.	$H \times B \times t$ (mm)	f'_c (MPa)	f_y (MPa)	N_e (kN)	N_e / N_c			
					EC4	ACI	AISC	Proposed model
C1-1	100×100×4.2	61	550	1490	1.07	1.13	1.14	1.05
C1-2	100×100×4.2	61	550	1535	1.10	1.16	1.17	1.08
C2-1	100×100×4.2	72	550	1740	1.17	1.24	1.26	1.17
C2-2	100×100×4.2	72	550	1775	1.19	1.27	1.28	1.19
C3	180×180×4.2	61	550	3590	1.05	1.14	1.15	1.06
C4	180×180×4.2	72	550	4210	1.12	1.23	1.24	1.16
C5-1	120×80×4.2	61	550	1450	1.06	1.12	1.14	1.04
C5-2	120×80×4.2	61	550	1425	1.04	1.10	1.12	1.02
C6-1	120×80×4.2	72	550	1560	1.07	1.14	1.16	1.07
C6-2	120×80×4.2	72	550	1700	1.16	1.24	1.26	1.17
C7-1	180×120×4.2	61	550	2530	1.01	1.08	1.10	1.01
C8-1	180×120×4.2	72	550	2970	1.09	1.18	1.20	1.11
C8-2	180×100×4.2	72	550	2590	0.95	1.03	1.05	0.97
C9-1	160×80×4.2	61	550	1710	0.99	1.05	1.08	1.02
C9-2	160×80×4.2	61	550	1820	1.05	1.11	1.15	1.08
C10-1	160×80×4.2	72	550	1880	1.02	1.08	1.12	1.06
C10-2	160×80×4.2	72	550	2100	1.13	1.21	1.25	1.18
C11-1	200×100×4.2	61	550	2350	0.97	1.04	1.07	1.03
C11-2	200×100×4.2	61	550	2380	0.98	1.05	1.08	1.04
C12-1	200×100×4.2	72	550	2900	1.11	1.20	1.23	1.19
C12-2	200×100×4.2	72	550	2800	1.07	1.16	1.19	1.15
Average					1.07	1.14	1.16	1.09
SD					0.07	0.07	0.07	0.07
COV					0.07	0.06	0.06	0.06

Table 4.3 Comparison of tests (FU), codes and proposed model

No.	$H \times B \times t$ (mm)	f'_c (MPa)	f_y (MPa)	N_e (kN)	N_e / N_c			
					EC4	ACI	AISC	Proposed model
R1-1	120×120×4	55	495	1739	1.08	1.16	1.16	1.06
R1-2	120×120×4	55	495	1718	1.07	1.14	1.15	1.05
R2-1	120×120×4	60	495	1701	1.02	1.09	1.10	1.01
R2-2	120×120×4	60	495	1657	0.99	1.06	1.07	0.98
R3-1	150×100×4	55	495	1815	1.08	1.16	1.17	1.07
R3-2	150×100×4	55	495	1763	1.05	1.12	1.14	1.04
R4-1	150×100×4	60	495	1735	1.00	1.07	1.08	0.99
R4-2	150×100×4	60	495	1778	1.02	1.09	1.11	1.02
R5-1	180×90×4	55	495	1725	0.95	1.02	1.04	0.99
R5-2	180×90×4	55	495	1742	0.96	1.03	1.05	1.00
R6-1	180×90×4	60	495	1773	0.94	1.01	1.03	0.98
R6-2	180×90×4	60	495	1795	0.95	1.02	1.05	0.99
R7-1	130×130×4	55	495	1963	1.08	1.16	1.17	1.07
R7-2	130×130×4	55	495	1988	1.09	1.17	1.18	1.08
R8-1	130×130×4	60	495	2020	1.07	1.15	1.16	1.06
R8-2	130×130×4	60	495	2018	1.07	1.15	1.16	1.06
R9-1	160×110×4	55	495	1947	1.03	1.10	1.12	1.02
R9-2	160×110×4	55	495	1912	1.01	1.08	1.10	1.00
R10-1	160×110×4	60	495	1982	1.01	1.08	1.10	1.01
R10-2	160×110×4	60	495	1923	0.98	1.05	1.07	0.98
R11-1	190×100×4	55	495	2035	1.00	1.07	1.09	1.03
R11-2	190×100×4	55	495	2138	1.05	1.13	1.15	1.08
R12-1	190×100×4	60	495	2049	0.97	1.04	1.06	1.00
R12-2	190×100×4	60	495	2124	1.00	1.08	1.10	1.03
R13-1	106×106×4	89	495	1749	1.05	1.14	1.15	1.10
R13-2	106×106×4	89	495	1824	1.10	1.19	1.20	1.14
R14-1	130×90×4	89	495	1752	1.01	1.10	1.11	1.06
R14-2	130×90×4	89	495	1806	1.04	1.13	1.15	1.09
R15-1	160×80×4	89	495	1878	0.99	1.08	1.11	1.06
R15-2	160×80×4	89	495	1858	0.98	1.06	1.09	1.05
R16-1	140×140×4	89	495	2752	1.05	1.15	1.16	1.11
R16-2	140×140×4	89	495	2828	1.08	1.18	1.19	1.14
R17-1	160×125×4	89	495	2580	0.96	1.06	1.07	1.02
R17-2	160×125×4	89	495	2674	1.00	1.09	1.11	1.06
Average					1.02	1.10	1.12	1.04
SD					0.05	0.05	0.05	0.04
COV					0.05	0.04	0.04	0.04

(a) Steel

(b) Concrete

Fig. 4.1 Proposed stress-strain relationships for CFSHS columns

Fig. 4.2 Effect of confinement factor on stress–strain relationship of concrete

Fig. 4.3 Effect of H/B ratio on stress–strain relationship of concrete

(a)

(b)

(c)

Fig. 4.4 Comparison of tests (NTU) and proposed model

(d)

(e)

(f)

Fig. 4.4 Comparison of tests (NTU) and proposed model (cont'd)

(g)

(h)

Fig. 4.4 Comparison of tests (NTU) and proposed model (cont'd)

Fig. 4.5 Comparison of tests (HIT) and proposed model

Fig. 4.5 Comparison of tests (HIT) and proposed model (cont'd)

Fig. 4.5 Comparison of tests (HIT) and proposed model (cont'd)

Fig. 4.5 Comparison of tests (HIT) and proposed model (cont'd)

Fig. 4.6 Comparison of tests (FU) and proposed model

Fig. 4.6 Comparison of tests (FU) and proposed model (cont'd)

Fig. 4.6 Comparison of tests (FU) and proposed model (cont'd)

Fig. 4.6 Comparison of tests (FU) and proposed model (cont'd)

Fig. 4.6 Comparison of tests (FU) and proposed model (cont'd)

(p)

(q)

Fig. 4.6 Comparison of tests (FU) and proposed model (cont'd)

Fig. 4.7 Calibration of proposed model against current test data

Fig. 4.8 Calibration of proposed model against previous test data

CHAPTER 5

EXPERIMENTAL STUDY OF RECTANGULAR CFSHS COLUMNS UNDER PURE BENDING

5.1 Introduction

This chapter presents an experimental study of the flexural behaviour of high-strength rectangular CFSHS columns. The primary objective of the experimental programme is to investigate the bending resistance and flexural stiffness of high-strength rectangular CFSHS columns. The bending resistance of composite columns is of significance for plotting axial force–bending moment interaction diagrams. Apart from bending resistance, flexural stiffness is also an important factor to consider for structural engineers, since the load distribution in a structure depends on the relative stiffness of the column and the beam. It should be noted that large discrepancies exist between the design codes, such as EC4 and ACI on estimating the flexural stiffness of composite columns. Therefore, the study of the moment resistance, flexural behaviour and ductility performance is carried out to assess the design codes and to propose design recommendations.

In the experimental study, a total of 12 high-strength rectangular CFSHS columns were constructed and tested to investigate the flexural behaviour. The test parameters were material strengths ($f_y=409\sim495$ MPa, $f'_c=56\sim91$ MPa) and cross-sectional aspect ratio (1.0~1.7). A cross-sectional view of the specimens is illustrated in Fig. 5.1. The specimens were tested under bending about the major axis. The applied load, strain and deflection of the specimens were recorded in the tests for analysis. The test procedure and results are presented and discussed in this chapter.

5.2 Material properties

The steel hollow sections for constructing the CFSHS specimens were cut from hot-finished structural steel shapes. Three sizes of hollow section were used, i.e. 150×150, 200×150 and 250×150 mm. The corner radius of the steel hollow section was measured as three times the wall thickness of the steel hollow section.

In order to determine the actual material properties of the steel hollow section, four coupons were flame cut from its four faces and machined according to the requirements in ASTM (1997). The coupons were tested under tension in an Instron testing machine at a loading rate of 0.5 mm per minute before the yield load and 1.5 mm per minute after the yield load. An extensometer with gauge length of 50 mm was clipped to the centre of the coupon to record its elongation during the test. The average yield stresses of the three types of steel sections (150×150, 200×150 and 250×150 mm) were determined as 438, 495 and 409 MPa, respectively. The results of the coupon tests are listed in Table 5.1. Typical stress–strain curves of the steel coupons are illustrated in Fig. 5.2.

The concrete used to fill the steel hollow sections was mixed in four batches with two types of mix design. The target compressive strengths of concrete with the two mix designs were 60 and 100 MPa, respectively. In order to achieve the desired high-strength concrete, the coarse aggregate was washed to remove the silt. From each batch of concrete mix, three concrete cubes (150×150×150 mm) and three cylinders (150×300 mm) were prepared. One day after the casting of concrete, the concrete samples were removed from the moulds and were cured in a fog room. At the same day as the CFSHS specimens were tested, the two end surfaces of the concrete cylinders were ground flat in a grinding machine. Then the concrete cubes and cylinders were crushed to failure in a 3000-kN-capacity compression machine to determine the actual material properties. The loading rates for testing concrete cubes and cylinders were 330 and 265 kN per minute, respectively. The average cylinder strengths of the four batches of concrete were determined as 56, 71, 88 and 91 MPa, respectively. The details of the mix proportions and the measured material properties of the four batches of concrete are summarised in Table 5.2.

5.3 Fabrication of specimens

The steel sections of the CFSHS specimens were cut from structural steel shapes and machined to a length of 1600 mm. A 10-mm-thick cap plate was welded to the base to support the wet concrete during casting. The concrete was vertically cast into the steel hollow section in four layers. Each layer of concrete was compacted using a poker vibrator. A 10-mm gap was left from the concrete surface to the top of the steel section. Fig. 5.3 gives a photograph of the 12 specimens after the completion of concrete casting. The specimen was cured in the laboratory with top open to the air for seven days for the concrete to set. Very little shrinkage was observed for the high-strength concrete infill. Subsequently, a layer of high-strength cement mortar was in-filled to flush the concrete core with the steel section. Finally, a 10-mm-thick cap plate was welded to the top to form a complete specimen. The geometric and material properties of the specimens are summarised in Table 5.3.

5.4 Test setup

The specimens were tested under pure bending about the major axis in the Heavy Structures Laboratory at NTU. As shown in Fig. 5.4, a 2000-kN-capacity hydraulic jack was fixed to a load frame to apply the vertical force to the specimen. Two concrete blocks were placed symmetrically about the hydraulic jack. The specimen was simply supported on the concrete blocks. The left roller support under the specimen allowed free rotation but restricted horizontal displacement, while the right roller support allowed free rotation and horizontal displacement. Same supports were placed at the quarter-length points (denoted as load points in Fig. 5.4) on the top flange of the specimen so that the load from the steel spreader beam can be transferred to the specimen. Alignment was performed to make sure that the centre of the specimen coincided with the centre of the spreader beam and the centre of the hydraulic jack. A plumb bob was employed for the alignment. A load cell was placed between the hydraulic jack and the spreader beam to record the applied load.

Four displacement transducers were used with two at the mid-span and the other two underneath the load points to measure the transverse deflection of the specimen

in the test. It should be noted that only the portion of the specimen between the two load points was subjected to pure bending. Thus, the mid-span deflection due to pure bending should be obtained by removing the displacement at the load points from the displacement at the mid-span. Four strain gauges were bonded in the longitudinal direction on the exterior surfaces of the steel section at the mid-span to determine the maximum compressive strain, the strain at the mid-height and the maximum tensile strain, as shown in Fig. 5.4(c).

5.5 Test procedure and results

The specimen was tested under monotonic bending about its major axis. In the test, the strain and the deflection of the specimen were first recorded at a load interval of 5% of the predicted failure load. Between each load level, the applied load was paused for a minute to allow the plasticity of the steel and cracking of the concrete to set. Sound of slippage and cracking of the concrete was first heard at a load level of approximately 50 kN for Specimens B01 to B04, 80 kN for Specimens B05 to B08 and 100 kN for Specimens B09 to B12. After the specimen had yielded, the load was applied continuously to capture the instantaneous response of the specimen. The deflection and strain values were taken at every 2 mm of the mid-span deflection. The test terminated at the maximum movement limit of the roller supports. The mid-span deflection upon the termination of the test was about 50 mm. For all specimens except B01 and B08, the deflection and strain readings were also recorded during the un-loading stage to examine the permanent deformation and strain.

5.5.1 Load–deflection relationship

The relationships of bending moment and mid-span deflection of the specimens are shown in Fig. 5.5. All the specimens exhibited significant yield plateau in the tests, indicating favourable ductility performance of the high-strength rectangular CFSHS columns in the post-yield region. Strain hardening was specifically noted for Specimens B09 to B12. For these four specimens, the applied load remained increasing in the tests. In Fig. 5.6, the bending moment is plotted against the deflection at the load point of the specimens.

5.5.2 Load–longitudinal strain relationship

Fig. 5.7 illustrates the correlation of the bending moment and maximum compressive strain, strain at the mid-height and maximum tensile strain for the specimens. As illustrated in the figure, both the compressive and tensile strains in the two flanges of the specimens were higher than 0.0025 at the yield plateau, indicating that both flanges of the specimen had yielded prior to the moment capacity. However, the mid-height of the webs was still within the elastic limit at the moment capacity. The strains at the mid-height of the specimens were lower than 0.0025, which revealed that the composite section did not fully develop its material plasticity at the moment capacity.

5.5.3 Flexural stiffness

Apart from the ultimate strength, flexural stiffness is also one of the research objectives for the pure bending tests on the composite columns. It should be noted that there has not been a widely accepted definition so far for defining the flexural stiffness of composite columns. The reason is that the cracking of concrete develops from the bottom flange to the top flange with the increase of bending moment. Consequently, the flexural stiffness of composite columns does not remain constant due to the propagation of cracks. In this experimental study, both the initial stiffness and the serviceability-level stiffness are investigated. The initial flexural stiffness of composite sections (K_{ini}) is defined to be the gradient of the moment–curvature curve at $M = 0.2M_e$, in which M_e is the experimental moment resistance (Varma et al. 2002a). The gradient of the moment–curvature curve at $M = 0.6M_e$ is defined to be the serviceability-level flexural stiffness (K_{ser}). Equations (5.1) and (5.2) are used for determining the flexural stiffness of the columns.

$$K = \frac{M}{\phi} \quad (5.1)$$

$$\phi = (\varepsilon_{top} - \varepsilon_{bot}) / H \quad (5.2)$$

in which, K is the flexural stiffness of composite columns, ε_{top} and ε_{bot} are the strain values in the top flange and bottom flange, respectively. The initial and serviceability-level flexural stiffnesses of the specimens are listed in Table 5.4.

5.5.4 Moment resistance

Since the applied load remained increasing in the post-yield region for most specimens, as shown in Fig. 5.5, it is not advisable to obtain the maximum bending moment as the bending resistance for the composite columns. In the current study, the moment capacity of the columns is determined as the bending moment at which the moment–deflection curve reaches a plateau. The moment capacities of the specimens are summarised in Table 5.4.

5.5.5 Failure modes

All the specimens exhibited favourable ductility performance in the tests. For Specimens B03, B04, B07, B08, B11 and B12, local buckling of the steel section was observed in the top flange at a mid-span deflection of about 40 mm. As shown in Fig. 5.8, local buckling occurred in the steel section close to the load point, instead of at the mid-span. Due to the local buckling, the specimen did not deform symmetrically about the mid-span point after the local buckling occurred.

In order to investigate the failure mode of the concrete, the steel hollow sections of Specimens B02, B04, B08 and B11 were flame cut after the bending tests. Fig. 5.9 gives photographs of the specimens after the steel sections were removed. It was observed that cracks of the concrete initiated from the tension side (bottom flange of the specimen) and propagated to about 2/3 of the height in the cross-section. For Specimens B04, B08 and B11, slight crushing of the concrete occurred at the position where local buckling of the steel section was observed. On the other hand, crushing of concrete was not observed for Specimen B02, which did not exhibit local buckling during the experiment. Despite the slight crushing of the concrete for some specimens, there was no significant drop of the applied load for all specimens in the tests. This was attributed to two factors. Firstly, the spalling and mass crushing of the concrete was avoided by the exterior steel section. Secondly, the volumetric steel-to-concrete ratios were as high as 0.12, which indicated that the steel section contributed to a predominant portion of the bending resistance of the composite section. Therefore, the specimens were able to exhibit a favourable post-yield behaviour although slip, cracking and minor crushing of the concrete occurred.

5.6 Concluding remarks

An experimental programme on high-strength rectangular CFSHS columns under pure bending has been presented. The moment resistance, ductility performance, flexural stiffness and failure mode were investigated by testing 12 specimens. The test procedure and results are described and discussed.

All the specimens manifested favourable ductility performance with significant yield plateau when subjected to pure bending. The test results suggested that the steel flanges of rectangular CFSHS columns developed material plasticity at the moment capacity, but the webs were still within elastic range.

Due to the propagation of cracks, the flexural stiffness of CFSHS columns subjected to pure bending did not remain constant. Both initial and serviceability-level flexural stiffnesses were defined and measured for the specimens tested in the current study. The experimental results are of significance to calibrate the specifications in the design codes for evaluating the flexural stiffness of high-strength CFSHS columns.

Typical failure mode of the high-strength rectangular CFSHS columns subjected to pure bending was associated with local buckling of the steel section in the top flange and minor crushing of the concrete. Since spalling and mass crushing of the concrete was prevented by the exterior steel hollow section, and the steel hollow section contributed a much larger proportion of the flexural strength to the composite section than the concrete core, slight crushing of the concrete inside the steel hollow section did not significantly affect the flexural behaviour of the high-strength rectangular CFSHS columns.

Table 5.1 Material properties of steel hollow sections

Steel section	Coupon no.	f_y (MPa)	f_u (MPa)	f_y/f_u	E_s (GPa)
150×150	1	433	517	0.84	207
	2	436	512	0.85	204
	3	430	509	0.85	204
	4	455	536	0.85	204
	Average	438	519	0.85	205
200×150	5	481	537	0.90	209
	6	496	544	0.91	205
	7	504	549	0.92	208
	8	498	545	0.91	209
	Average	495	544	0.91	208
250×150	9	391	509	0.77	206
	10	418	521	0.80	208
	11	408	509	0.80	209
	12	418	528	0.79	208
	Average	409	517	0.79	208

Table 5.2 Details of high-strength concrete

Mix design no.	A		B	
Cement (ordinary portland cement) (kg/m ³)	480		550	
Silica fume (kg/m ³)	20		40	
Water (kg/m ³)	170		165	
Maximum aggregate size (mm)	20		10	
Coarse aggregate (kg/m ³)	1025		950	
Sand (kg/m ³)	665		661	
Water-reducing admixture (ml/m ³)	5760		4720	
Superplasticizing admixture (ml/m ³)	0		5900	
Batch no.	1	2	3	4
Average density (kg/m ³)	2376	2386	2400	2438
Modulus of elasticity (GPa)	38	41	43	44
Average cube strength (MPa) at 48 days	78	89	103	107
Average cylinder strength (MPa) at 48 days	56	71	88	91

Table 5.3 Geometric and material properties of specimens

Specimen no.	H (mm)	B (mm)	t (mm)	H/B	B/t	f'_c (MPa)	f_y (MPa)	α
B01	150	150	4.9	1.0	31	56	438	0.13
B02	150	150	4.9	1.0	31	56	438	0.13
B03	150	150	4.9	1.0	31	88	438	0.13
B04	150	150	4.9	1.0	31	88	438	0.13
B05	200	150	5.9	1.3	34	56	495	0.13
B06	200	150	5.9	1.3	34	56	495	0.13
B07	200	150	5.9	1.3	34	88	495	0.13
B08	200	150	5.9	1.3	34	88	495	0.13
B09	250	150	5.9	1.7	42	71	409	0.12
B10	250	150	5.9	1.7	42	71	409	0.12
B11	250	150	5.9	1.7	43	91	409	0.12
B12	250	150	5.9	1.7	43	91	409	0.12

Table 5.4 Results of pure bending tests

Specimen	Section	f_y (MPa)	f'_c (MPa)	M_e (kNm)	K_{ini} (kNm ²)	K_{ser} (kNm ²)
B01	150×150×4.9	438	56	81	2536	1734
B02	150×150×4.9	438	56	85	2259	1769
B03	150×150×4.9	438	88	99	2579	2092
B04	150×150×4.9	438	88	101	2886	2140
B05	200×150×5.9	495	56	164	6409	4819
B06	200×150×5.9	495	56	170	6218	4922
B07	200×150×5.9	495	88	178	6379	5087
B08	200×150×5.9	495	88	175	6619	5135
B09	250×150×5.9	409	71	211	11084	10002
B10	250×150×5.9	409	71	208	11447	10040
B11	250×150×5.9	409	91	216	11955	10359
B12	250×150×5.9	409	91	216	12214	10467

Fig. 5.1 Cross-sectional view of rectangular CFSHS specimen

Fig. 5.2 Stress-strain curves of steel hollow sections

Fig. 5.3 Specimens in curing

(a) Specimen in the test rig

(b) Diagrammatic view

(c) Section A-A: strain gauge locations

Fig. 5.4 Test setup for specimens subjected to pure bending

Fig. 5.5 Moment–deflection relationships at mid-span

Fig. 5.5 Moment–deflection relationships at mid-span (cont'd)

Fig. 5.6 Moment–deflection relationships at load point

Fig. 5.6 Moment-deflection relationships at load point (cont'd)

Fig. 5.7 Moment-strain relationships of specimens

Fig. 5.7 Moment-strain relationships of specimens (cont'd)

Fig. 5.7 Moment-strain relationships of specimens (cont'd)

Fig. 5.7 Moment-strain relationships of specimens (cont'd)

Fig.5.8 Local buckling of specimens

(a)

(b)

(c)

(d)

(e)

Fig. 5.9 Failure modes of concrete core

CHAPTER 6

ANALYTICAL STUDY OF RECTANGULAR CFSHS COLUMNS UNDER PURE BENDING

6.1 Introduction

This chapter presents an analytical study of the flexural behaviour of high-strength rectangular CFSHS columns. The primary objective of this study is to assess the specifications in the design codes and to develop an analytical model for predicting the behaviour of high-strength rectangular CFSHS columns subjected to pure bending.

Section 6.2 reviews the design philosophies in the codes (EC4, ACI and AISC) pertaining to composite columns subjected to pure bending. The methods in the design codes for estimating the moment resistance and flexural stiffness are evaluated. Section 6.3 describes the constitutive relationships employed in the analysis for simulating the stress–strain responses of steel and concrete. Section 6.4 presents an analytical model which is constructed for predicting the flexural behaviour of high-strength CFSHS columns. The assumptions made in the development of the analytical model are also given. In Section 6.5, the experimental results given in Chapter 5 are used to calibrate the proposed analytical model, as well as the specifications in the design codes. Both the ultimate capacity and the flexural stiffness of these specimens are considered for calibration. Based on experimental and analytical results, recommendations are proposed in this section for the design of high-strength rectangular CFSHS columns under pure bending. In Section 6.6, the test data reported by other researchers are employed to calibrate the design codes and the proposed analytical model. Section 6.7 presents the conclusions of this analytical study of the flexural behaviour of high-strength rectangular CFSHS columns.

6.2 Code specifications

Various philosophies are adopted in the design codes EC4, ACI and AISC for the design of CFSHS columns subjected to pure bending, partly because the composite action between the steel and the concrete has not been adequately understood. This section reviews the code specifications for the design of composite columns under pure bending.

According to EC4, the moment resistance of a CFSHS column is determined based on following assumptions (a) the steel section and the concrete infill can fully develop their material strengths f_y and f'_c respectively at the moment capacity; (b) there is full strain compatibility between the steel section and the concrete infill; (c) concrete tensile strength is omitted in the calculation and (d) the concrete cylinder strength and the steel yield stress are not higher than 50 and 355 MPa, respectively.

To estimate the flexural stiffness of composite column, the following equation is specified in EC4.

$$EI = E_s I_s + 0.6 E_c I_c \quad (6.1)$$

ACI evaluates the flexural strength of a CFSHS column based on the assumptions that (a) the plane cross-section of the column before bending remains plane during bending; (b) the maximum compressive strain of concrete at the moment capacity is 0.003; (c) the maximum compressive stress of concrete is $0.85 f'_c$ and (d) the tensile strength of concrete is neglected.

To estimate the flexural stiffness of composite columns, Equation (6.2) is recommended in ACI.

$$EI = E_s I_s + 0.2 E_c I_c \quad (6.2)$$

AISC predicts the flexural strength of a CFSHS column by assuming that (a) plastic stress distribution exists in the cross-section and (b) there is no composite action between the steel and the concrete. Therefore, the moment resistance of the cross-section is determined based on the steel hollow section alone.

To estimate the flexural stiffness of composite columns, Equation (6.3) is stated in AISC.

$$EI = (E_s + 0.4 E_c A_c / A_s) I_s \quad (6.3)$$

Since the material properties of the specimens tested in the current study are known, the material safety factors specified in the codes are set to unity in the calculation of the flexural strength. Moreover, the resistance factors specified in the codes are also set to unity in the calculation.

6.3 Constitutive relationships for steel and concrete

In order to simulate the flexural behaviour of high-strength rectangular CFSHS columns, an analytical model is constructed in the study. The development of the analytical model encompasses proposing respective constitutive relationships for steel and concrete and constructing a computer programme for the analysis.

The constitutive relationship for steel is identical to the stress–strain model developed in Chapter 4, as given in Equation (4.10). This four-branch model incorporates the elastic, plastic and strain hardening behaviours of the steel material. For the high-strength concrete confined by rectangular hollow sections, a material model is constructed in the study, as shown in Fig. 6.1. The stress–strain relationship of concrete in compression is determined using Equation (6.4). Strength increase of the concrete resulting from confinement effect is considered.

$$\sigma_c = f_0 \frac{1.6x}{0.6 + x} \quad \text{for } x \leq 1 \quad (6.4a)$$

$$\sigma_c = f_0 \quad \text{for } x > 1 \quad (6.4b)$$

in which,

$$f_0 = 0.85 f'_c \eta$$

$$\eta = 1 + \frac{[1 - 0.65(\beta - 1)^3](0.37 \xi^{0.82} - 0.19 \xi)}{1 + 0.05(f'_c / 50)^{5.65}}$$

$$\xi = A_s f_y / (0.85 A_c f'_c)$$

$$\beta = H / B, \text{ cross-sectional aspect ratio } (B \leq H)$$

$$x = \varepsilon_c / \varepsilon_0$$

ε_c = strain of concrete

$$\varepsilon_0 = 1300 + (1300 + 18 f'_c) \xi^{0.2}$$

For concrete in tension, a stress–strain relationship proposed by Hsu (1998) is modified and used in the numerical analysis. The modified model for simulating the tensile behaviour of confined concrete is shown in Equation (6.5).

$$\sigma_c = E_c \varepsilon_c \quad \text{for } \varepsilon_c \leq \varepsilon_t \quad (6.5a)$$

$$\sigma_c = f_t (\varepsilon_t / \varepsilon_c)^{0.4} \quad \text{for } \varepsilon_c > \varepsilon_t \quad (6.5b)$$

in which, f_t is the tensile strength of concrete, which can be determined as in Equation (6.6) (Nawy 2003). It should be noted that f'_c and f_t in Equation (6.6) are in MPa.

$$f_t = 0.61 \sqrt{f'_c} \quad (6.6)$$

$$\varepsilon_t = f_t / E_c \quad (6.7)$$

6.4 Development of analytical model

A numerical model is constructed in the study to simulate the flexural behaviour of rectangular CFSHS columns. This model consists of a cross-sectional analysis based on the proposed constitutive relationships and a global analysis based on a deflected shape imposed to the column, as shown in Fig. 6.2. The following assumptions are made in developing the model: (a) the deflection of the column under pure bending has a shape of sine curve; (b) plane cross-section before bending remains plane during bending and (c) there is no slip between the steel section and the concrete infill.

Based on the assumptions, the deflection of the middle part of the specimen, which is subjected to pure bending, can be expressed as in Equation (6.8).

$$u = u_m \sin\left(\frac{x\pi}{0.5L_0}\right) \quad (6.8)$$

in which, L_0 is the effective length of specimen; u_m is the mid-span deflection due to pure bending.

For a given mid-span deflection of u_m , the curvature of the specimen can be determined as in Equation (6.9).

$$\phi = \pi^2 u_m / (0.5L_0)^2 \quad (6.9)$$

If the composite section is discretized as in Fig. 6.2(b), the finite strip at a height of y from the geometric centre of the cross-section has the strain of

$$\varepsilon_i = \varepsilon_{CR} + y_i \phi \quad (6.10)$$

in which ε_{CR} is the strain at the geometric centre of the cross-section.

The stresses of steel and concrete strips can thus be determined based on respective constitutive relationships. The axial force acting on the composite section can be determined as in Equation (6.11).

$$N = \sum(\sigma_{Ci} dA_{Ci} + \sigma_{Si} dA_{Si}) \quad (6.11)$$

Since the column is subjected to pure bending, the axial force should be zero. Based on this, the strain at the geometric centre of the cross-section (ε_{CR}) can be determined in a trial-and-error manner for the given mid-span deflection.

The bending moment at this given mid-span deflection can be determined as in Equation (6.12).

$$M = \sum(\sigma_{Ci} y_i dA_{Ci} + \sigma_{Si} y_i dA_{Si}) \quad (6.12)$$

A computer programme is developed based on the equations mentioned above to predict the moment–deflection relationship. Fig. 6.3 shows the flow chart of the computer programme. This numerical procedure can produce the load–deflection relationship for experimentally studied columns. Therefore, it provides a useful comparison between theoretical and experimental results. The numerical procedure employed in the current study is referenced to the work of Susantha et al. (2001), Uy (2000), Han (2004a) and Sakino et al. (2004).

6.5 Calibration of proposed model with present test data

6.5.1 Moment capacity

The moment capacities obtained in the tests are compared with the calculated values using the design specifications in the codes and the proposed analytical model. As summarised in Table 6.1, the design codes and the analytical model are conservative to estimate the moment capacities of the specimens. Among the codes, EC4 gives the closest prediction with an average N_e/N_c ratio of 1.09. It should be noted that the assumption of plastic stress distribution is made in EC4. Therefore, the strength predicted by EC4 is the upper bound of the moment capacity. However, current tests show that the EC4 approach is still conservative to predict the flexural strengths of the columns. One possible reason can be attributed to the corner effect of the steel hollow sections. On the other hand, ACI gives conservative estimates of the ultimate capacities of the specimens by 15%. Since the concrete core is neglected by AISC both in determining the neutral axis and in calculating the flexural strength of the composite section, AISC significantly underestimates the flexural strengths by 32%. The proposed model, however, provides close estimates of the flexural strengths with an average difference of 9%. The comparison of results indicates that the model can be used for estimating the moment capacity of high-strength rectangular CFSHS columns with reasonable accuracy.

6.5.2 Moment–deflection relationship

The comparison of moment–deflection relationship between the tests and the proposed model is given in Fig. 6.4. Since the specimen is subjected to four-point loading, only the middle part of the specimen is under pure bending. Thus the mid-span deflection recorded from the displacement transducers is not solely due to pure bending. Equation (6.13) is used to obtain the mid-span deflection due to pure bending.

$$u_m = u_1 - u_0 \quad (6.13)$$

in which, u_m is the mid-span deflection due to pure bending, u_1 is the overall mid-span deflection, u_0 is the deflection at the load point.

6.5.3 Flexural stiffness

The measured initial flexural stiffnesses of the specimens are compared with the calculated values using the design codes and the proposed analytical model. As summarised in Table 6.2, EC4 overestimates the initial flexural stiffness by 6%. The ACI approach, however, gives conservative estimates of the initial flexural stiffness by 17%. The AISC method is un-conservative to predict the stiffness by 14%. The proposed model also overestimates the initial flexural stiffness of the specimens with an average difference of 7%. It should be mentioned that slip is not considered in the proposed analytical model. Since slip can occur between the steel section and the concrete infill when the composite column is subjected to bending, the experimental flexural stiffness is lower than that predicted by the proposed analytical model.

The experimental serviceability-level flexural stiffnesses are also compared with the analytical results, as summarised in Table 6.3. Comparison suggests that all the three design codes are un-conservative to estimate the serviceability-level flexural stiffness. Among the codes, ACI gives closest estimate of the stiffness by 6%. EC4 and AISC over-predict the serviceability-level flexural stiffness by 25 and 31%, respectively. The proposed model also overestimates the serviceability-level flexural stiffness of the specimens by 16%. The overestimation of the proposed analytical model can be attributed to the omission of slip in the analysis.

After the discussion mentioned above, it is concluded that EC4 is the most suitable approach to estimate the initial flexural stiffness for high-strength rectangular CFSHS columns. On the other hand, ACI can be recommended for predicting the serviceability-level flexural stiffness. Due to the defect that slip is not considered in the proposed model, close estimate of the flexural stiffness using the model is not obtained.

6.6 Calibration of proposed model with previous test data

In order to calibrate the design codes and the proposed model in a wide range of variables, previous tests data reported by Tomii (1979), Lu and Kennedy (1994), Uy (2000, 2001a) and Han (2004) are referenced. A total of 40 specimens with $f_y = 194 \sim 750$ MPa, $f'_c = 19 \sim 47$ MPa and $\alpha = 0.04 \sim 0.19$ were tested in the five research

projects. The comparison of the moment capacities between the tests and the analytical results is summarised in Table 6.4. For ease of comparison, the concrete cylinder (100×200 mm) strengths reported in Tomii (1979) and Uy (2000, 2001b) are converted to the equivalent cylinder (150×300 mm) strengths using a conversion factor of 0.90, as suggested by Martinez et al. (1984).

Comparison of results suggests that the proposed model underestimates the moment capacities of the specimens with an average difference of 7%. The provisions in EC4, ACI and AISC are also conservative to predict the flexural strengths of the specimens by 5, 13 and 24%, respectively. It can be concluded that the EC4 approach can be used for estimating the flexural strength of rectangular CFSHS columns. However, it should be noted that the codes cannot predict the moment–deflection relationship for CFSHS columns. Hence, the proposed model is of significance for predicting the flexural behaviour of high-strength rectangular CFSHS columns.

6.7 Concluding remarks

An analytical study of the flexural behaviour of high-strength rectangular CFSHS columns has been described. A numerical model is constructed to predict the moment–deflection relationship and the ultimate capacity of composite columns under pure bending. In the development of the analytical model, the stress–strain relationship of the confined concrete is proposed.

The test data obtained in the experimental programme are employed to calibrate the proposed analytical model. The comparison of moment–deflection relationships between the tests and the proposed model demonstrates that the model is capable of accurately predicting the flexural behaviour of high-strength rectangular CFSHS columns. The comparison of moment capacities is also carried out between the tests, the design codes and the proposed model. Results suggest that both EC4 and the proposed model are able to yield close and conservative estimates of the flexural strength for the CFSHS columns.

The flexural stiffness values between the tests, design codes and the analytical model are also compared. Results indicate that the EC4 method gives the closest

estimates of the initial flexural stiffness, whilst ACI yields the closest prediction of the serviceability-level flexural stiffness for the specimens.

The previous test data pertaining to rectangular CFSHS columns under pure bending are also employed to calibrate the proposed analytical model and the design codes. Comparison shows that the analytical model underestimates the moment capacities with an average difference of 7%. Since the design codes cannot be used to estimate the moment–deflection relationship, the proposed model is important for the analytical study of the flexural behaviour of CFSHS columns.

Based on the research findings in the analytical study, recommendations are highlighted for the design of high-strength rectangular CFSHS columns subjected to pure bending.

Table 6.1 Comparison of flexural strength between tests, codes and proposed model

Specimen	M_e (kNm)	EC4		ACI		AISC		Proposed model	
		M_c (kNm)	M_e/M_c	M_c (kNm)	M_e/M_c	M_c (kNm)	M_e/M_c	M_c (kNm)	M_e/M_c
B01	81	79	1.02	75	1.08	68	1.19	79	1.03
B02	85	79	1.07	75	1.13	68	1.25	79	1.08
B03	99	81	1.23	78	1.27	68	1.46	82	1.21
B04	101	81	1.25	78	1.29	68	1.49	82	1.23
B05	164	163	1.01	149	1.10	137	1.20	158	1.04
B06	170	163	1.04	149	1.14	137	1.24	158	1.08
B07	178	166	1.07	156	1.14	137	1.30	163	1.09
B08	175	166	1.06	156	1.12	137	1.28	163	1.07
B09	211	195	1.08	183	1.15	157	1.34	194	1.09
B10	208	195	1.07	183	1.14	157	1.32	194	1.07
B11	216	198	1.09	189	1.14	157	1.38	200	1.08
B12	216	198	1.09	189	1.14	157	1.38	200	1.08
Average			1.09		1.15		1.32		1.09
SD			0.07		0.06		0.09		0.06
COV			0.07		0.05		0.07		0.06

Table 6.2 Comparison of initial flexural stiffness between tests, codes and proposed model

Specimen	K_{ini} (kNm ²)	EC4		ACI		AISC		Proposed model	
		K_c (kNm ²)	K_{ini}/K_c	K_c (kNm ²)	K_{ini}/K_c	K_c (kNm ²)	K_{ini}/K_c	K_c (kNm ²)	K_{ini}/K_c
B01	2536	2782	0.91	2293	1.11	3098	0.82	2832	0.90
B02	2259	2782	0.81	2293	0.99	3098	0.73	2832	0.80
B03	2579	2879	0.90	2325	1.11	3236	0.80	3015	0.86
B04	2886	2879	1.00	2325	1.24	3236	0.89	3015	0.96
B05	6409	6582	0.97	5416	1.18	7133	0.90	6505	0.99
B06	6218	6582	0.94	5416	1.15	7133	0.87	6505	0.96
B07	6379	6813	0.94	5492	1.16	7436	0.86	6910	0.92
B08	6619	6813	0.97	5492	1.21	7436	0.89	6910	0.96
B09	11084	12079	0.98	9526	1.24	12924	0.91	12140	0.97
B10	11447	12079	0.95	9526	1.20	12924	0.89	12140	0.94
B11	11955	12359	0.97	9619	1.24	13266	0.90	12771	0.94
B12	12214	12359	0.99	9619	1.27	13266	0.92	12771	0.96
Average			0.94		1.17		0.86		0.93
SD			0.05		0.08		0.06		0.05
COV			0.05		0.07		0.07		0.06

Table 6.3 Comparison of serviceability-level flexural stiffness between tests, codes and proposed model

Specimen	K_{ser} (kNm ²)	EC4		ACI		AISC		Proposed model	
		K_c (kNm ²)	K_{ser}/K_c	K_c (kNm ²)	K_{ser}/K_c	K_c (kNm ²)	K_{ser}/K_c	K_c (kNm ²)	K_{ser}/K_c
B01	1734	2782	0.62	2293	0.76	3098	0.56	2543	0.68
B02	1769	2782	0.64	2293	0.77	3098	0.57	2543	0.70
B03	2092	2879	0.73	2325	0.90	3236	0.65	2648	0.79
B04	2140	2879	0.74	2325	0.92	3236	0.66	2648	0.81
B05	4819	6582	0.73	5416	0.89	7133	0.68	5857	0.82
B06	4922	6582	0.75	5416	0.91	7133	0.69	5857	0.84
B07	5087	6813	0.75	5492	0.93	7436	0.68	6112	0.83
B08	5135	6813	0.75	5492	0.93	7436	0.69	6112	0.84
B09	10002	12079	0.83	9526	1.05	12924	0.77	10570	0.95
B10	10040	12079	0.83	9526	1.05	12924	0.78	10570	0.95
B11	10359	12359	0.84	9619	1.08	13266	0.78	10863	0.95
B12	10467	12359	0.85	9619	1.09	13266	0.79	10863	0.96
Average			0.75		0.94		0.69		0.84
SD			0.07		0.11		0.08		0.10
COV			0.10		0.12		0.11		0.11

Table 6.4 Comparison of previous tests, codes and proposed model

Ref.	Specimen no.	$B \times H \times t$ (mm)	f'_c (MPa)	f_y (MPa)	M_e (kNm)	M_e/M_c			
						EC4	ACI	AISC	Proposed model
Tomii (1979)	I-0	100×100×2.29	22.1	198	7.3	0.95	0.97	1.12	0.94
	II-0	100×100×2.27	19.8	311	11.2	0.95	1.00	1.11	0.97
	III-0	100×100×2.98	18.9	295	14.3	1.00	1.06	1.15	1.03
	IV-0	100×100×4.25	17.1	290	18.7	0.95	1.05	1.10	1.02
Lu and Kennedy (1994)	CB12	152×152×4.8	47.0	389	73.6	1.14	1.17	1.37	1.13
	CB13	152×152×4.8	42.8	389	75.1	1.11	1.12	1.34	1.09
	CB15	152×152×4.8	41.2	389	71.3	1.15	1.18	1.38	1.15
	CB22	152×152×9.5	46.9	432	146.5	1.06	1.07	1.30	1.04
	CB31	254×152×6.4	46.7	377	210.7	0.99	1.00	1.23	0.97
	CB33	254×152×6.4	45.2	377	210.7	0.90	1.23	1.06	1.02
	CB35	254×152×6.4	44.3	377	207.6	0.89	1.16	1.04	0.98
	CB41	254×152×9.5	46.2	394	283.8	0.83	1.04	0.96	0.89
	CB45	254×152×6.4	43.8	394	282.2	1.04	1.09	1.21	1.05
	CB52	152×254×6.4	47.1	377	144.7	1.06	1.12	1.24	1.08
	CB53	152×254×6.4	42.1	377	146.7	1.01	1.06	1.17	1.03
	CB55	152×254×6.4	40.5	377	142.9	1.02	1.13	1.17	1.08
Uy (2000)	HS6	126×126×3	45	300	27.9	1.07	1.16	1.31	1.11
	HS12	156×156×3	45	300	42.4	1.07	1.16	1.31	1.11
	NS6	186×186×3	28.8	300	62.6	1.06	1.15	1.29	1.10
	NS12	246×246×3	34.2	300	103.5	0.96	1.09	1.17	1.04
	NS18	246×246×3	34.2	300	153	0.96	1.09	1.16	1.04
Uy (2001a)	HSS6	110×110×5	27.0	750	66	1.15	1.17	1.29	1.15
	HSS13	160×160×5	27.0	750	141	1.17	1.20	1.30	1.17
	HSS19	210×210×5	28.8	750	228	1.14	1.17	1.27	1.14
Han (2004)	1	120×120×3.84	21.5	330	29.3	0.99	1.06	1.14	1.00
	2	120×120×3.84	27.6	330	30.2	1.02	1.07	1.18	1.01
	3	120×120×3.84	27.6	330	32.3	1.09	1.15	1.26	1.08
	4	120×120×3.84	27.6	330	31.7	1.07	1.13	1.23	1.06
	5	120×120×5.86	25.9	321	40.9	0.96	1.05	1.08	1.00
	6	120×120×5.86	25.9	321	41.5	0.98	1.07	1.10	1.01
	7	120×120×5.86	31.5	321	41.4	0.98	1.05	1.10	1.00
	8	120×120×5.86	31.5	321	42.6	1.01	1.08	1.13	1.02
	9	150×120×2.93	27.2	294	31.4	1.08	1.13	1.30	1.04
	10	150×120×2.93	27.2	294	31.5	1.08	1.13	1.30	1.04
	11	150×90×2.93	27.2	294	29.3	1.20	1.27	1.44	1.19
	12	150×90×2.93	27.2	294	29.4	1.15	1.22	1.37	1.13
	13	120×90×2.93	27.2	294	21.1	1.14	1.22	1.39	1.14
	14	120×90×2.93	27.2	294	20.2	1.18	1.26	1.44	1.18
	15	120×60×2.93	27.2	294	18.4	1.27	1.43	1.57	1.35
	16	120×60×2.93	27.2	294	17.8	1.23	1.38	1.52	1.31
Av						1.05	1.13	1.24	1.07
SD						0.10	0.09	0.13	0.09
COV						0.09	0.08	0.11	0.08

Fig. 6.1 Stress-strain model for confined concrete

Fig. 6.2 Sectional analysis of CFSHS specimens

Fig. 6.3 Flow chart of the computer programme

Fig. 6.4 Comparison of measured and predicted moment–deflection relationships

Fig. 6.4 Comparison of measured and predicted moment–deflection relationships
(cont'd)

CHAPTER 7

EXPERIMENTAL STUDY OF RECTANGULAR CFSHS COLUMNS UNDER ECCENTRIC LOADING

7.1 Introduction

This chapter presents two experimental programmes carried out to investigate the behaviour of high-strength rectangular CFSHS columns subjected to eccentric loading. A total of 16 stub and 16 slender CFSHS columns fabricated from high-strength materials were tested under eccentric loading about the major axis. The objective of testing stub columns under eccentric loading was to study the effect of load eccentricity on the strength of CFSHS columns. On the other hand, slender columns were tested so that the effects of both the slenderness and the load eccentricity on the ultimate capacity of CFSHS columns can be investigated.

Section 7.2 describes a set of eccentric loading tests carried out at Fuzhou University (FU) on 20 high-strength rectangular CFSHS columns. The test variables were material strengths, slenderness ratio (10 and 60) and load eccentricity ratio ($e/H = 0.10\sim 0.42$). The test procedure and results of the 20 columns are given in detail in this section. The effects of load eccentricity and slenderness on the ultimate capacity, as well as the failure mode of the columns are also discussed.

Section 7.3 presents an experimental study of 12 high-strength CFSHS slender columns carried out at Harbin Institute of Technology (HIT). The test parameters were material strengths, slenderness ratio (20 and 50) and load eccentricity ratio ($e/H = 0.10\sim 0.40$). The test procedure and results are presented in detail. The effects of load eccentricity and slenderness on the ultimate capacity, as well as the failure mode of the specimens are discussed in this section.

Section 7.4 summarises the research findings from these two experimental programmes.

7.2 Experimental programme at FU

A total of 20 rectangular CFSHS columns were constructed and tested at FU. Among the specimens, 16 were stub columns with $L/H=3$ while the remaining four were slender specimens with slenderness ratio of 60. Test parameters of the specimens were material strengths ($f_y=495$ MPa; $f'_c=60$ MPa), cross-sectional aspect ratio ($\beta=1.0\sim2.0$), load eccentricity ratio ($e/H=0.10\sim0.42$) and slenderness ratio ($\lambda=10$ and 60). All these parameters were selected according to the ranges of practical limits in the construction industry.

7.2.1 Material properties

The steel hollow section was fabricated by welding its four component flat plates together, as shown in Fig. 3.1(b). All the flat plates were cut from a 1400-mm-wide steel coil. In order to determine the actual material properties, three coupons were cut from the steel coil and machined according to the specifications in ASTM (1997). The steel coupons were tested in tension in an Instron testing machine at a loading rate of 0.5 mm per minute before the failure load and 2.0 mm per minute after the failure load. An extensometer with gauge length of 50 mm was clipped to the centre of the steel coupons to record the elongation during the test. The average yield stress, tensile strength and modulus of elasticity were determined as 495, 581 and 206,000 MPa, respectively. The results of the tension tests on the steel coupons are listed in Table 3.6. A typical stress–strain curve of the steel obtained from the coupon tests is illustrated in Fig. 3.4(c).

The concrete for the 20 composite columns was mixed in one batch. From the concrete mix, four concrete cubes ($150\times150\times150$ mm) and four cylinders (150×300 mm) were prepared. Two days after the casting of concrete, the concrete samples were stripped from the moulds and were exposed to the air in the laboratory. Water was sprayed to the concrete samples twice a day to compensate for the moisture lost due to evaporation. At the same day as the CFSHS columns were tested, both ends of the concrete cylinders were ground flat using an electric disk grinder. Then the concrete cubes and cylinders were crushed in a 3,000-kN-capacity testing machine to determine the actual material properties. The loading rate for the concrete cubes was approximately 330 kN per minute, whilst for cylinders it was about 265 kN per

minute. The average cylinder strength and the modulus of elasticity of the concrete were determined as 60 and 39,000 MPa, respectively. Table 3.7 (Batch B) summarises the details of the mix proportion and the measured material properties of the concrete.

7.2.2 Fabrication of specimens

A CFSHS column consisted of a welded steel hollow section, two cap plates and the concrete infill. After the welding of the steel hollow section was completed, an 8-mm-thick cap plate was welded to its base to hold the wet concrete during casting. Then concrete was vertically cast into the steel hollow section in layers. Each layer was compacted by a poker vibrator. A 10-mm gap was left from the concrete surface to the top of the steel section. The specimen was subsequently cured in the laboratory with top open to the air for 14 days for the concrete to set. Subsequently, a layer of high-strength cement mortar was in-filled to flush the concrete core with the steel section. Finally, the other cap plate was welded to the top to form a complete specimen. The geometric and material properties of the specimens are summarised in Table 7.1.

7.2.3 Test setup

The specimens were tested under eccentric loading about the major axis in a 5000-kN-capacity universal testing machine in the Structures Laboratory at FU. Since the standard accessories of the testing machine could not produce eccentric loading, special end assemblages were constructed which allowed the load from the testing machine to be applied at given eccentricities to the specimen. Fig. 7.1 gives a photograph and a detailed drawing of the end assemblages. As shown in this figure, the end assemblages were able to produce eccentricities at 5 mm intervals from 10 to 60 mm. The end assemblages, so-called knife edges were employed at both the bottom and top of the specimen to simulate pin-pin boundary conditions.

The knife edges were made from high-strength steel and were stiff enough to avoid deformation during the tests. The knife edges consisted of a male edge and a female counterpart. A short steel rebar with a diameter of 20 mm and a height of 10 mm was welded to the centre of the cap plates. The female edge had a hole of 22 mm in diameter and 15 mm in depth at the centre to accommodate the steel rebar of

the specimen. The steel rebar was useful for the alignment of the specimen before the test.

Fig. 7.2 illustrates the test setup for the stub columns. The deflection in the plane of loading was measured by a displacement transducer at mid-length ($L/2$). Four strain gauges were bonded on the exterior surfaces of the specimen at $L/2$ to record longitudinal strains.

Fig. 7.3 shows the test setup for the slender columns. The transverse deflection in the plane of loading was recorded by three displacement transducers at heights of $0.25L$, $0.5L$ and $0.75L$, respectively. The out-of-plane deflection of the specimen was monitored by another displacement transducer at $L/2$. Four strain gauges were bonded on the four exterior surfaces of the specimen at $L/2$ to measure longitudinal strains.

7.2.4 Test procedure and results

The specimen was tested to failure under eccentric loading about the major axis. Before the test, the bottom male edge was placed at the centre of the bottom platten of the testing machine. Then the bottom female edge was positioned onto the male edge according to the required load eccentricity. The specimen was subsequently placed on the male edge. Finally, the top female and male edges were placed to the top of the specimen. Alignment was performed to ensure that the centres of the male edges coincided with the centres of the platens of the testing machine. Prior to the actual test, a load level of 20 kN was applied so that the platens of the testing machine and the knife edges firmly contacted the specimen. Then the axial load was applied slowly to the specimen by careful manipulation of the loading and unloading valves. During the test, the load, strain and deflection readings of the specimen were recorded at a load increment of 50 kN.

For the stub columns, the displacement transducer was only employed to measure the deflection for Specimens S1~S4, not for the other stub columns due to safety considerations. This was because it was feared that the specimen may slip from the testing machine when the load eccentricity was large. Therefore, only the longitudinal strains were measured for the other stub columns. It should be noted that the deflection of stub columns was only for reference and cannot be used for calculation, because the slenderness effect was negligible for stub columns. On the

other hand, transverse deflection readings were important for slender columns because of the second-order effect. During the tests, the out-of-plane deflection of the slender specimen was found to be less than 1 mm, thus the specimen was confirmed to be under axial compression combined with uniaxial bending moment.

Fig. 7.4 gives the correlation of the axial force and the longitudinal strains for the stub columns. As shown in this figure, when the failure load was reached, the longitudinal strain in the compression flange (Strain Gauge 1) was higher than 0.0025, indicating the yielding of steel material. The strain at the mid-depth ($H/2$) of the cross-section (Strain Gauge 2) was higher than 0.0025 except Specimens S3, S8, S14 and S16. This suggested that the steel section from the compression flange to the centre of the web had yielded before the failure load for most specimens. The strain in the tension flange (Strain Gauge 3) however, was lower than 0.0025 for all stub columns. For stub columns with load eccentricity (e/H) ratio less than 0.13, i.e. Specimen S1, S5 and S9, the tension flange was still in compression and within the elastic range at the failure load. For all the other stub columns with $e/H = 0.20 \sim 0.42$, the tension flange was in tension but also within the elastic range at the failure load. These observations reveal that stub CFSHS columns with $e/H = 0.10 \sim 0.42$ cannot fully develop material plasticity at the failure load. Hence, plastic assumption will not be suitable for the analysis of high-strength rectangular CFSHS stub columns subjected to eccentric loading.

The relationships of the applied load and the longitudinal strains for the slender columns are given in Fig. 7.5. As shown in this figure, the compression flange (Strain Gauge 1) experienced material yielding of the steel section. The centre of the web (Strain Gauge 2) was still within the elastic range for all the four specimens. On the other hand, the stress state of the tension flange (Strain Gauge 3) at the failure load was found to be influenced by the e/H ratio. For Specimens L1 ($e/H = 0.10$) and L2 ($e/H = 0.20$), the tension flange was still within the elastic range at the failure load. However, for Specimens L3 ($e/H = 0.30$) and L4 ($e/H = 0.40$), the tension flange experienced steel yielding at the failure load.

Fig. 7.6(a) shows the load–deflection relationships for the stub columns, whilst Fig. 7.6(b) illustrates the load–deflection relationships for the slender columns. As shown in this figure, the axial load decreased slowly after the failure load,

indicating the favourable ductility performance of the columns in the post-yield region. It can be concluded from the comparison of behaviours between the four slender columns that the load eccentricity had an adverse effect on the strength of the columns. The experimental ultimate axial load (N_e) and the ultimate end bending moment ($M_e = N_e e$) for the 20 specimens are summarised in Table 7.1.

Two types of failure mode were observed for the stub columns in the tests. Fig. 7.7 gives a photograph of all the stub columns after the compression tests. The first type of failure was identified as the local buckling in the compression flange and the two adjacent webs. The other type of failure was identified as local buckling in the compression flange only. It was also noted that the position of local buckling was affected by the load eccentricity ratio (e/H). For specimens with $e/H \geq 0.40$, local buckling occurred close to the ends of the specimens, such as S8 and S16. On the other hand, for all the other stub columns with $e/H < 0.40$, local buckling occurred at heights of approximately $L/4$. With the decrease of e/H ratio, local buckling tended to move from close to the ends to the mid-length ($L/2$). This also indicates that for stub columns with high load eccentricity ratios, failure may occur at the two ends. Therefore, measures should be taken so that end effect can be excluded. Otherwise, the axial load capacity of the composite column cannot be obtained due to failure at the ends.

In order to investigate the failure mode of the concrete in the stub columns, the steel sections of Specimens S1, S5 and S9 were flame cut after the tests. As shown in Fig. 7.8, concrete slightly crushed at the position where the steel hollow section locally buckled. Mass crushing and spalling of the concrete were not observed due to the protection and confinement by the exterior steel hollow section. Moreover, the rigid concrete infill prevented the steel hollow section from buckling inward. This explains the favourable ductility performance of the CFSHS stub columns in the eccentric loading tests.

All the four slender columns performed in a much similar failure mode to each other. As illustrated in Fig. 7.9, the specimens buckled in the plane of loading. Local buckling of steel section was observed about 100 mm above or below the mid-length of the specimens.

7.3 Experimental programme at HIT

A total of 12 rectangular CFSHS columns were constructed and tested in the Structures Laboratory of HIT. The test parameters of the specimens were material strengths ($f_y = 550$ MPa, $f'_c = 61$ and 72 MPa), cross-sectional aspect ratio ($\beta = 1.0 \sim 2.0$), volumetric steel-to-concrete ratio ($\alpha = 0.12 \sim 0.20$), load eccentricity ratio ($e/H = 0.17 \sim 0.40$) and slenderness ratio ($\lambda = 20$ and 50). All these parameters were determined according to the ranges of practical limits in the construction industry.

7.3.1 Material properties

The steel hollow sections were fabricated by welding four component flat plates together, as shown in Fig. 3.13. All the component steel plates were cut from a 1600-mm-wide steel coil, which was manufactured according to the requirements in ASTM (1997) for Steel A572 Grade 65. Its nominal yield stress and tensile strength were 450 and 550 MPa, respectively. In order to obtain the actual material properties, six steel coupons were cut from the steel coil. Among the six steel coupons, three were machined according to the requirements in ASTM (1997), whilst the other three conformed to the specifications in the Chinese testing standards GB2975 (1982). All the steel coupons were tested in an Instron testing machine at loading rates of 0.5 and 1.5 mm per minute before and after the yield load, respectively. The measured yield stress, tensile strength and modulus of elasticity are summarised in Table 3.4. As the discrepancies of results between the two codes were minimal, the measured values determined according to GB2975 were used in the study. The average yield stress, tensile strength and modulus of elasticity were determined as 550, 608 and 207,000 MPa, respectively.

The concrete for the 12 specimens was mixed in two batches. From each batch of concrete, three concrete cubes ($150 \times 150 \times 150$ mm) were prepared. Due to the lack of cylindrical moulds in the lab, concrete cylinders were not constructed. The moulds were stripped from the concrete samples one day after the concrete was cast. Then the concrete samples were cured in the lab, exposed to the air. Water was sprayed to the concrete cubes once a day to compensate for the moisture loss due to evaporation. At the same day as the CFSHS specimens were tested, the concrete cubes were crushed to determine the actual compressive strengths. The average

cube strengths of the two batches of concrete were determined as 71 and 82 MPa, respectively. For the ease of using the code specifications in the analytical study, the cube strengths were converted into the equivalent cylinder strengths following the table proposed by Chen et al. (1996). The equivalent cylinder strengths of concrete were determined as 61 and 72 MPa, respectively. The moduli of elasticity of the two batches of concrete were determined as 39,000 and 40,000 MPa, respectively.

7.3.2 Fabrication of specimens

After the welding of the steel hollow section was completed, the two end surfaces of the steel hollow sections were ground flat in a lathe. A 10-mm-thick base cap was welded to the steel hollow section so as to hold the wet concrete during casting. Then, concrete was vertically cast into the steel hollow section in layers, each of which was compacted using a poker vibrator. A 10-mm gap was left from the concrete surface to the top of the steel section. The specimen was cured in the lab with top open to the air for two weeks for the concrete to set. A layer of high-strength cement mortar was subsequently in-filled to flush the concrete core with the steel hollow section. Finally, the top cap plate was welded to form a complete specimen. The geometric and material properties of the 12 specimens are given in Table 7.2.

7.3.3 Test setup

The specimens were tested in a 5000-kN-capacity universal testing machine in the Structures Laboratory of HIT. Fig. 7.10(a) gives a front view of a specimen in the testing machine. Since the standard accessories of the testing machine could not produce eccentric loading, knife edges were constructed so that the compressive load could be applied to the specimen at given load eccentricities. A schematic view of the test setup is given in Fig. 7.10(b). Knife edges were implemented at both the bottom and the top of the specimen to simulate pin-pin boundary conditions. The knife edges consisted of a male edge and a female counterpart. The male edge was bolted to the top and bottom platens of the testing machine. The female edge was positioned to the top and the bottom cap plates of the specimen. A 20-mm-in-diameter and 10-mm-in-height steel rebar was welded to the centre of the cap plates

in order to align the female edges to the specimen and to prevent the slip between the female edges and the specimen. During the test, the deflection in the plane of loading over the length of the specimen was recorded by five displacement transducers. The deflection of the specimens out of the loading plane was monitored by three displacement transducers. The displacement of the two platens of the testing machine was also monitored by two displacement transducers. 12 strain gauges were bonded on the steel section to measure longitudinal and transverse strains, as shown in Fig. 7.10(c).

7.3.4 Test procedure and results

The specimens were tested to failure under eccentric loading about the major axis. Before the test, the two male edges were bolted to the upper and lower platens of the testing machine. The bottom female edge was positioned onto the bottom male edge according to the required load eccentricity. The specimen was subsequently placed on the female edge. Finally, the top female edge was placed onto the specimen. Prior to the actual test, a load level of 20 kN was applied so that the two platens of the testing machine, knife edges and the specimen firmly contacted each other. The axial load was then slowly applied to the specimen by careful manipulation of the loading and unloading valves. The longitudinal and the transverse strains as well as the in-plane and out-of-plane deflections of the specimen were recorded at a load increment of 50 kN. The experimental ultimate axial load (N_e) and the ultimate end bending moment ($M_e = N_e e$) are summarised in Table 7.2. During the tests, the out-of-plane deflections of all specimen were less than 1 mm, thus the specimens were confirmed to be under compression combined with uniaxial bending.

The axial force–deflection relationships of the 12 specimens are illustrated in Fig. 7.11. As shown in this figure, the axial force decreased slowly after the failure load, indicating the favourable ductility performance of the specimens in the post-yield region. It can be concluded from the comparison of behaviours between the specimens, such as specimens E01 and E02 that the strength of concrete had a positive effect on the ultimate capacity of the columns. On the other hand, the slenderness and load eccentricity demonstrated an adverse influence on the strength of the columns.

In Fig. 7.12, the axial force is plotted against the strain values of the compression and tension flanges in the mid-length cross-section. The strain in the compression flange of all specimens was higher than 0.003 at the failure load, indicating the yielding of the steel material. For the columns with slenderness ratio of 20, the tension flange was still within the elastic range at the failure load. On the other hand, for columns with slenderness ratio of 50, the stress state in the tension flange at the failure load was dependent on the e/H ratio. The slender columns with $e/H=0.38\sim0.42$ experienced material yielding in the tension flange, whilst the tension flange of other slender columns with $e/H=0.17\sim0.20$ was still within the elastic range.

Three failure modes were observed for the specimens in the tests. Fig. 7.13 gives photographs for all the specimens after the eccentric loading tests. The first type of failure, which occurred for Specimens E01 and E06, was identified as crushing of concrete and local buckling of steel hollow section close to the end of the specimen. The second type of failure mode occurred for all the other specimens with slenderness ratio of 20. Local buckling was observed near the quarter-length of the specimen. The third type of failure mode was noticed for all the specimens with slenderness ratio of 50. Local buckling occurred in the compression flange of the steel section. Besides, the specimens buckled in a symmetric shape about the mid-length.

7.4 Concluding remarks

Two experimental programmes have been described in this chapter on the behaviour high-strength rectangular CFSHS columns subjected to eccentric loading. The ultimate capacity, ductility performance and the failure mode were extensively investigated by testing 32 stub and slender columns. The test procedure and results are presented and discussed.

The test results demonstrated favourable ductility performance for all specimens. The adverse effects of slenderness and load eccentricity on the ultimate capacity of CFSHS beam-columns were observed in the experiments.

The experiments showed that the failure mode of CFSHS beam-columns was influenced by the slenderness and the load eccentricity. For stub columns, local

buckling occurred close to the quarter-length. On the other hand, overall buckling was observed for slender columns.

The tests also manifested that eccentrically loaded high-strength rectangular CFSHS columns cannot fully develop material strengths at the failure load. Hence, the assumption of plastic stress distribution will not be suitable for estimating the ultimate capacity. The test data reported herein are of great significance for calibrating the specifications in the codes for the design of high-strength CFSHS columns. Moreover, the test data can also be employed to verify analytical models for evaluating the behaviour of high-strength rectangular CFSHS columns subjected to eccentric loading.

Table 7.1 Summary of specimens tested at FU

No.	$H \times B \times t$ (mm)	H/B	L (mm)	λ	α	e (mm)	e/H	N_e (kN)	M_e (kNm)
S1	120×120×4	1.0	360	10	0.15	15	0.13	1294	19.4
S2	120×120×4	1.0	360	10	0.15	25	0.21	1125	28.1
S3	120×120×4	1.0	360	10	0.15	30	0.25	949	28.5
S4	120×120×4	1.0	360	10	0.15	45	0.38	810	36.5
S5	150×100×4	1.5	450	10	0.15	15	0.10	1422	21.3
S6	150×100×4	1.5	450	10	0.15	30	0.20	1190	35.7
S7	150×100×4	1.5	450	10	0.15	45	0.30	964	43.4
S8	150×100×4	1.5	450	10	0.15	60	0.40	763	45.8
S9	180×90×4	2.0	540	10	0.15	20	0.11	1491	29.8
S10	180×90×4	2.0	540	10	0.15	30	0.17	1319	39.6
S11	180×90×4	2.0	540	10	0.15	40	0.22	1208	48.3
S12	180×90×4	2.0	540	10	0.15	50	0.28	1051	52.6
S13	130×130×4	1.0	390	10	0.14	15	0.12	1472	22.1
S14	130×130×4	1.0	390	10	0.14	25	0.19	1305	32.6
S15	130×130×4	1.0	390	10	0.14	40	0.31	1022	40.9
S16	130×130×4	1.0	390	10	0.14	55	0.42	789	43.4
L1	150×100×4	1.5	2600	60	0.15	15	0.10	1130	17.0
L2	150×100×4	1.5	2600	60	0.15	30	0.20	884	26.5
L3	150×100×4	1.5	2600	60	0.15	45	0.30	711	32.0
L4	150×100×4	1.5	2600	60	0.15	60	0.40	617	37.0

Note: $f_y = 495$ MPa; $f'_c = 60$ MPa.

Table 7.2 Summary of specimens tested at HIT

No.	$H \times B \times t$ (mm)	H/B	L (mm)	f'_c (MPa)	α	λ	e (mm)	e/H	N_e (kN)	M_e (kNm)
E01	150×150×4.2	1.0	870	61	0.12	20	30	0.20	1678	50.3
E02	150×150×4.2	1.0	870	72	0.12	20	30	0.20	1850	55.5
E03	150×150×4.2	1.0	2170	61	0.12	50	30	0.20	1330	39.9
E04	150×150×4.2	1.0	2170	72	0.12	50	60	0.40	1020	61.2
E05	180×120×4.2	1.5	1040	61	0.13	20	30	0.17	1950	58.5
E06	180×120×4.2	1.5	1040	72	0.13	20	70	0.39	1140	79.8
E07	120×80×4.2	1.5	1740	61	0.20	50	20	0.17	660	13.2
E08	120×80×4.2	1.5	1740	72	0.20	50	20	0.17	855	17.1
E09	200×100×4.2	2.0	1150	61	0.14	20	60	0.30	1310	78.6
E10	200×100×4.2	2.0	1150	72	0.14	20	40	0.20	1800	72.0
E11	160×80×4.2	2.0	2310	61	0.18	50	60	0.38	670	40.2
E12	160×80×4.2	2.0	2310	72	0.18	50	30	0.19	1020	30.6

Note: $f_y = 550$ MPa.

(a) Photograph

(b) Schematic view: female edge

(c) Schematic view: male edge

Fig. 7.1 Details of knife edges

(a) Specimen in the test rig

(b) Schematic view

(c) Section A-A: strain gauge locations

Fig. 7.2 Test setup for stub columns at FU

(a) Photograph

(b) Schematic view

(c) Section A-A: strain gauge locations

Fig. 7.3 Test setup for slender columns at FU

(a) Specimen S1

(b) Specimen S2

(c) Specimen S3

Fig. 7.4 Load-strain relationships for stub columns tested at FU

(d) Specimen S4

(e) Specimen S5

(f) Specimen S6

Fig. 7.4 Load-strain relationships for stub columns tested at FU (cont'd)

(g) Specimen S7

(h) Specimen S8

(i) Specimen S9

Fig. 7.4 Load-strain relationships for stub columns tested at FU (cont'd)

(j) Specimen S10

(k) Specimen S11

(l) Specimen S12

Fig. 7.4 Load-strain relationships for stub columns tested at FU (cont'd)

(m) Specimen S13

(n) Specimen S14

(o) Specimen S15

Fig. 7.4 Load-strain relationships for stub columns tested at FU (cont'd)

(p) Specimen S16

Fig. 7.4 Load–strain relationships for stub columns tested at FU (cont'd)

(a) Specimen L1

(b) Specimen L2

(c) Specimen L3

Fig. 7.5 Load-strain relationships for slender columns tested at FU

(d) Specimen L4

Fig. 7.5 Load–strain relationships for slender columns tested at FU (cont'd)

(a) Stub columns

(b) Slender columns

Fig. 7.6 Load-deflection relationships for specimens tested at FU

Fig.7.7 Failure modes of stub columns tested at FU

Fig.7.8 Failure modes of concrete of stub columns tested at FU

(a) Local buckling of specimens during tests

(b) Specimens after tests

Fig.7.9 Failure modes of slender columns tested at FU

(a) Photograph

(b) Schematic view

(c) Section A-A: strain gauge locations

Fig. 7.10 Test setup for columns at HIT

(a) Specimens with slenderness ratio of 20

(b) Specimens with slenderness ratio of 50

Fig.7.11 Load-deflection relationships for specimens tested at HIT

(a) Specimens with slenderness ratio of 20

(b) Specimens with slenderness ratio of 50

Fig.7.12 Load–strain relationships for specimens tested at HIT

Fig.7.13 Failure modes of specimens tested at HIT

CHAPTER 8

ANALYTICAL STUDY OF RECTANGULAR CFSHS COLUMNS UNDER ECCENTRIC LOADING

8.1 Introduction

This chapter provides an analytical study of the behaviour of high-strength rectangular CFSHS columns subjected to eccentric loading. The philosophies adopted in the codes (EC4, ACI and AISC) for the design of CFSHS columns subjected to combined axial loading and bending moment are presented in Section 8.2. The discrepancies between the design codes and their limitations for the design of CFSHS columns are discussed in this section.

An analytical model is presented in Section 8.3 for estimating the behaviour of eccentrically loaded high-strength rectangular CFSHS columns. The development of constitutive relationships for steel and concrete and the assumptions made in the analytical model are discussed.

In Section 8.4, the proposed analytical model is calibrated against the experimental results of the 32 columns obtained in the experimental study presented in Chapter 7. The comparison of ultimate capacity and load–deflection response between the tests and the proposed model is discussed in detail. The specifications in the design codes are also employed to estimate the ultimate capacities of the specimens and compared with the test results.

In order to calibrate the proposed analytical model against test data in wide ranges, the experimental results reported by other researchers are also employed for the verification, as presented in Section 8.5. The test results of 60 CFSHS columns are referenced for the verification of the proposed analytical model. Finally, research findings of this analytical study are summarised in Section 8.6.

8.2 Code specifications

In this section, the design philosophies adopted in the codes EC4, ACI and AISC for the design of CFSHS columns under axial loading combined with bending moment are discussed.

8.2.1 Eurocode 4, EC4 (1994)

According to EC4, the resistance of a composite section to eccentric loading can be determined based on the following assumptions: (a) both the steel section and the concrete have fully developed their material plasticity at the ultimate capacity; (b) there is full strain compatibility between the steel section and the concrete core; (c) the compressive strength of concrete is taken as f'_c and (d) the tensile strength of concrete is neglected.

The resistance of the composite section can be determined from its axial force–bending moment interaction diagram. Since the determination of the interaction diagram requires comprehensive calculation, usually only achievable with computer programmes, it is not suitable for practical design. Therefore, a linear approximation to the interaction curve is also recommended in EC4, as expressed by Lines ABCD in Fig. 8.1. Points A, B, C, and D can be determined according to the stress blocks specified in Fig. 8.2. A quick calculation on the resistance of a composite section at a given axial load level or load eccentricity can be achieved by linear interpolation between Points A, B, C and D.

In order to determine the resistance of a slender composite column, the slenderness effect should be incorporated. Second-order analysis may apply in this situation, as specified in EC4. In order to simplify the estimation of the slenderness effect, a moment amplification method is also recommended in EC4. The maximum bending moment in the column can be obtained by applying a magnifier factor to the end bending moment. Equation (8.1) shows the moment amplification factor specified in EC4 for CFSHS columns under axial compression combined with equal end bending moments.

$$k = \frac{1.1}{1 - N / N_{eu}} \quad (8.1)$$

in which,

N = applied axial force

$N_{eu} = \pi^2 EI / L^2$, Euler buckling load of column

$EI = E_s I_s + 0.6 E_c I_c$

I_s = moment of inertia of steel hollow section

I_c = moment of inertia of un-cracked concrete core

8.2.2 American Concrete Institute, ACI (2002)

According to ACI, the resistance of a composite section is evaluated based on the following assumptions: (a) a linear strain distribution exists in the cross-section; (b) there is no slip between the steel and the concrete; (c) the maximum allowable compressive strain of concrete is 0.003; (d) the maximum compressive stress in the concrete is $0.85 f'_c$ and (e) the tensile strength of concrete is omitted.

Similar to EC4, a moment amplification method is specified in ACI for estimating the slenderness effect. For a slender composite column under axial loading combined with equal end bending moments, a moment amplification factor as in Equation (8.2) applies to incorporate the second-order effect.

$$k = \frac{1}{1 - N / (0.75 N_{eu})} \quad (8.2)$$

in which, $N_{eu} = \pi^2 EI / L^2$, $EI = E_s I_s + 0.2 E_c I_c$.

8.2.3 American Institute of Steel Construction, AISC (1999)

The AISC approach for the design of CFSHS columns is essentially identical to that for steel columns. The resistance of a composite section is determined from the axial force–bending moment interaction diagram, as in Equation (8.3).

$$\frac{N}{N_0} + \frac{8M}{9M_0} = 1.0 \quad \text{for } \frac{N}{N_0} \geq 0.2 \quad (8.3a)$$

$$\frac{N}{2N_0} + \frac{M}{M_0} = 1.0 \quad \text{for } \frac{N}{N_0} < 0.2 \quad (8.3b)$$

in which,

$$N_0 = A_s f_{cr}$$

$$f_{cr} = (0.658^{\lambda_c^2}) f_{my} \text{ for } \lambda_c \leq 1.5$$

$$f_{cr} = (0.877 / \lambda_c^2) f_{my} \text{ for } \lambda_c > 1.5$$

$$f_{my} = f_y + 0.85 f'_c A_c / A_s$$

$$\lambda_c = \frac{L}{r_s \pi} \sqrt{f_{my} / (E_s + 0.4 E_c A_c / A_s)}$$

$$r_s = \text{radius of gyration of steel hollow section}$$

In Equation (8.3), the nominal flexural strength M_0 is the cross-sectional moment capacity at zero axial load level. It is determined based on plastic stress distribution for both the steel and the concrete in the cross-section. However, the extent to which the concrete core contributes to the cross-sectional moment resistance depends on the axial load level. For $N / N_0 \geq 0.3$, full composite action is assumed, while for $N / N_0 < 0.3$, M_0 is determined by straight line transition between the nominal strength at $N / N_0 = 0.3$ and the nominal strength at $N = 0$. It should be noted that the nominal strength at $N = 0$, i.e. the pure bending capacity is determined based on the steel section only. The concrete is not considered in either determining the neutral axis or calculating the moment capacity. This statement in AISC makes the calculation of M_0 at various axial load levels not consistent. This is because according to the linear transition statement for determining M_0 at $N / N_0 = 0.0 \sim 0.3$, when $N / N_0 = 0$, the concrete core should be considered in determining the position of neutral axis, but not considered in final calculation of the bending moment. The complex specifications in AISC may be attributed to the lack of understanding of the bond strength and its effect on the behaviour of composite columns subjected to bending moment.

The determination of resistance of a slender composite column requires that the slenderness effect be taken into account. AISC also adopts a moment magnifier method for this purpose. For composite columns under axial loading combined with equal end bending moments, a factor as in Equation (8.4) applies to consider the second-order effect.

$$k = \frac{1}{1 - N / N_{eu}} \quad (8.4)$$

in which, $N_{eu} = \pi^2 E_m I_s / L^2$; $E_m = E_s + 0.4 E_c A_c / A_s$. It is noted that the slenderness effect is considered twice in the AISC approach, i.e. in both Equation (8.3) and Equation (8.4).

8.3 Proposed analytical model

In order to predict the behaviour of high-strength rectangular CFSHS columns under eccentric loading, an analytical model is constructed in the study. The development of the analytical model includes cross-sectional analysis using proposed constitutive relationships and global analysis based on a deflected shape imposed to the column. A computer programme is constructed for this numerical procedure.

8.3.1 Constitutive relationships for steel and concrete

The four-branch constitutive relationship as shown in Equation (4.10) is developed and used for simulating the stress–strain response of the steel. This constitutive model incorporates the elastic, plastic and strain hardening behaviours of the steel material.

For high-strength concrete confined by rectangular steel sections, a constitutive relationship as in Equation (8.5) is developed in the analytical study for simulating the stress–strain behaviour of concrete in compression. For concrete in tension, the tensile strength of concrete is not considered in the calculation. The concrete model is shown in Fig. 8.3.

$$\sigma_c = E_c \varepsilon_c \quad \text{for } \varepsilon_c \leq \varepsilon_1 \quad (8.5a)$$

$$\sigma_c = f_0 \quad \text{for } \varepsilon_1 < \varepsilon_c \leq \varepsilon_2 \quad (8.5b)$$

$$\sigma_c = f_0 (\varepsilon_2 / \varepsilon_c)^{0.24+0.45/\xi} \quad \text{for } \varepsilon_c > \varepsilon_2 \quad (8.5c)$$

in which,

$$f_0 = 0.85 f_c' \eta$$

$$\eta = 1 + \frac{[1 - 0.65(\beta - 1)^3] (0.37 \xi^{0.82} - 0.19 \xi)}{1 + 0.05(f_c' / 50)^{5.65}}$$

$$\xi = A_s f_y / (0.85 A_c f_c')$$

$$\beta = H / B$$

$$\varepsilon_1 = f_0 / E_c$$

$$\varepsilon_2 = 2 \varepsilon_1$$

8.3.2 Development of analytical model

An analytical model is developed in the study for predicting the structural behaviour of eccentrically loaded CFSHS columns. The proposed analytical model includes the cross-sectional analysis, which is based on the material constitutive relationships and the global analysis, which is based on the following assumptions: (a) the deflection of the columns has a shape of sine curve; (b) the plane cross-section before loading remains plane during loading; (c) there is no slip between the steel section and the concrete and (d) the tensile strength of concrete is not considered. A computer programme is developed for the numerical analysis of composite columns. The analytical model developed in the present study is referenced to the work reported by Vrcelj and Uy (2002), Mursi and Uy (2003), Han (2004a) and Sakino et al. (2004) for predicting the behaviour of composite columns subjected to eccentric loading.

Based on the assumptions, the transverse deflection of an eccentrically loaded column can be expressed as

$$u = u_m \sin(x\pi / L) \quad (8.6)$$

in which, u_m is the transverse deflection at $L/2$, L is the buckling length.

At a given u_m , the curvature of the column can be determined as

$$\phi = \pi^2 u_m / L^2 \quad (8.7)$$

If the composite section is discretized as in Fig. 8.4(b), the finite strip at a height of y_i from the geometric centre of the cross-section has the strain of

$$\varepsilon_i = \varepsilon_{CR} + y_i \phi \quad (8.8)$$

in which, ε_{CR} is the strain at the geometric centre of the composite section.

The stresses of the steel and concrete in this finite strip can thus be determined based on the strain and respective constitutive relationships. Based on the areas and stresses of the steel and concrete, the axial force and the bending moment acting on the composite section can be determined as in Equations (8.9) and (8.10), respectively.

$$N = \sum(\sigma_{Ci} dA_{Ci} + \sigma_{Si} dA_{Si}) \quad (8.9)$$

$$M = \sum(\sigma_{Ci} y_i dA_{Ci} + \sigma_{Si} y_i dA_{Si}) \quad (8.10)$$

Since the specimen is subjected to eccentric loading with a given eccentricity, the following equation should be satisfied.

$$M / N = u_m + e \quad (8.11)$$

Based on Equation (8.11), the strain at the centre of the cross-section for a given mid-height deflection can be determined in a trial-and-error manner. A computer programme is constructed and employed to predict the axial force–deflection ($N-u_m$) relationship of the eccentrically loaded CFSHS columns. Fig. 8.5 shows the flow chart of the computer programme. The maximum axial force of the column at a given load eccentricity can be obtained by running this computer programme and plotting the $N-u_m$ curve.

8.4 Calibration of proposed model with present test data

8.4.1 Force–deflection relationship

The analytical model is employed to predict the force–deflection relationships of the 32 specimens tested in the study as given in Chapter 7. Fig. 8.6 shows the comparison of the proposed analytical model and the experimental results of the four slender columns tested at FU. It suggests that the proposed analytical model agrees very well with the test curves for Specimens L1, L2 and L4. Relative large

discrepancy exists between the experimental and analytical $N-u_m$ responses for Specimen L3, however. The mid-height deflection of the test is larger than that predicted by the numerical model. One possible reason accounting for this discrepancy is the geometric imperfection of this column.

The proposed analytical model is also calibrated against the tests results obtained at HIT, as shown in Fig. 8.7. Results suggest that the proposed model is able to predict the $N-u_m$ responses for most specimens with reasonable accuracy. It is noted that the experimental mid-height deflections are larger than those predicted by the analytical model. This can be attributed to two factors. First of all, the specimens with $\lambda=20$ did not buckle in a shape of sine curve. As described in Chapter 7, local buckling occurred at $L/4$ for the specimens with $\lambda=20$, instead of at $L/2$. The other reason is that the displacement transducer at $L/2$ was placed at the compression side, instead of the tension side due to safety considerations. One problem may arise in this situation. Local buckling may occur close to the place where the displacement transducer contacted the specimen. The readings were the sum of the deflection and the local buckling, not the actual deflection of the column.

8.4.2 Ultimate capacity

The analytical model, as well as the design codes is calibrated against the failure loads of the 32 specimens tested in the experimental study. The comparison of the model with the test results of the 16 stub columns tested at FU is summarised in Table 8.1. SD and COV in Table 8.1 denote the standard deviation and coefficient of variation, respectively. Comparison indicates that EC4 is un-safe to predict the ultimate capacity of the 16 stub columns. The average experimental-to-calculated failure loads (N_e/N_c) ratio is 0.97, while the coefficient of variation is 0.06. The minimum and maximum N_e/N_c ratios are 0.84 and 1.02, respectively. This implies that the assumption of plastic stress distribution made in EC4 is not suitable for high-strength composite columns under eccentric loading. On the other hand, both ACI and AISC are conservative to assess the ultimate capacities of the columns. ACI safely predicts the failure loads by 13% with a coefficient of variation of 0.03. The minimum and maximum N_e/N_c ratios are 1.05 and 1.17, respectively. This reveals that the assumption of the strain limit of 0.003 for the concrete at the ultimate capacity is too strict for high-strength CFSHS columns. At this strain limit,

only the out-most steel in the cross-section can yield. The AISC provisions, on the other hand significantly underestimate the strengths of the specimens by 24%. The minimum and maximum N_e/N_c ratios are 1.17 and 1.30, respectively. Since the design equations in AISC are essentially identical to those for steel columns, modifications are thus required so that the contribution of the concrete core to the moment resistance of the composite section can be more reasonably considered. Compared with the three design codes, the proposed model yields a more accurate calculation on the failure loads with an average difference of 3%. The coefficient of variation is 0.05. The minimum and maximum N_e/N_c ratios are 0.93 and 1.12, respectively.

Table 8.2 illustrates the verification of the proposed model and the design codes with the test results of the four slender columns tested at FU. Results show that the EC4 approach is un-conservative to estimate the ultimate capacities by 7%. The minimum and maximum N_e/N_c ratios are 0.89 and 0.97, respectively. The overestimation of EC4 indicates that the plastic stress distribution may not occur for high-strength slender CFSHS columns under eccentric loading. Therefore, the EC4 approach should be modified before it can be used with confidence for the design of high-strength CFSHS columns. The ACI and AISC specifications, on the other hand conservatively estimate the failure loads by 19% and 23%, respectively. The moment amplification factors for the four columns according to the design codes are also listed in Table 8.2. It shows large discrepancies between the design codes on how to consider the slenderness effect. In contrast, the proposed model gives much closer estimates of the failure loads with an average difference of 1%. The minimum and maximum N_e/N_c ratios are 1.00 and 1.02, respectively.

Table 8.3 summarises the calibration of the proposed analytical model and the codes with the test data of the 12 slender columns tested at HIT. Results suggest that EC4 is un-conservative to estimate the strengths of the 12 slender columns with an average difference of 2%. The minimum and maximum N_e/N_c ratios are 0.81 and 1.15, respectively. The ACI method on the other hand conservatively estimates the failure loads by 17%. The minimum and maximum N_e/N_c ratios are 0.95 and 1.30, respectively. The AISC specifications significantly underestimate the ultimate capacities of the 12 specimens by 25%. The minimum and maximum N_e/N_c ratios

are 0.99 and 1.41, respectively. In comparison with the design codes, the proposed model gives closer estimates of the failure loads with an average difference of 4%. The minimum and maximum N_e/N_c ratios are 0.95 and 1.20, respectively.

Axial force–bending moment interaction diagrams are also constructed and compared with the test data of the 16 stub columns to shed some light on the discrepancy between the tests, the design codes and the proposed model. As shown in Fig. 8.8, EC4 is able to give close estimates of the failure loads for specimens with load eccentricity ratios (e/H) less than 0.20. However, with the increase of e/H ratio, it tends to yield un-conservative results. ACI is conservative for all the 16 specimens because of its maximum compressive strain (0.003) assumption. Due to the less accurate consideration of the composite action, AISC is over conservative for the high-strength composite columns. Revisions to the three codes are required before an accurate and economical design can be achieved. The proposed analytical model, however, produces very close estimates of the failure loads for the specimens, as shown in Figs. 8.8 and 8.9. This can be attributed to the abolition of the maximum strain limit for the concrete, reasonable consideration of the material stress–strain behaviour and comprehensive second-order analysis which incorporates the slenderness effect.

8.5 Calibration of proposed model with previous test data

The proposed analytical model is also verified with the test data pertaining to eccentrically loaded rectangular CFSHS columns reported in the literature, i.e. Han (2000), Uy (2001a), Varma et al (2002a), Han and Yao (2003) and Fujimoto et al (2004). The test variables of the 60 specimens in these research projects are in wide ranges with $f_y=262\sim824$ MPa, $f'_c=18\sim110$ MPa, $H/B=1.0\sim2.0$ and $\alpha=0.06\sim0.21$.

As shown in Table 8.4, the proposed analytical model closely predicts the ultimate capacities of the 60 specimens with an average difference of 3%. The minimum and maximum experimental-to-predicted failure loads ratios are 0.84 and 1.16, respectively. Fig. 8.10 shows the comparison of the test data and the analytical model. It can be concluded that the proposed model is capable of predicting the

ultimate capacity of both high-strength and normal strength rectangular CFSHS columns subjected to combined axial loading and bending moment.

8.6 Concluding remarks

An analytical study has been presented on the behaviour of high-strength rectangular CFSHS columns subjected to eccentric loading. An analytical model is developed to estimate the structural behaviour of rectangular CFSHS beam-columns. the proposed analytical model is calibrated against the test data obtained in the experimental programme described in Chapter 7 and those reported by other researchers. The specifications in the design codes pertaining to the design of composite columns are also considered and verified with the test data.

The comparison of failure loads between the design codes and the current test data suggests that EC4 is un-safe for the design of high-strength CFSHS beam-columns. For the slender columns tested at HIT, the average difference is 2%. For the stub and slender columns tested at FU, the average differences are 3% and 7%, respectively. This implies that the rigid-plastic assumption made in EC4 is not suitable for high-strength composite columns. On the other hand, the ACI approach is conservative compared with the test results. For the slender columns tested at HIT, the average difference is 17%, whilst for the stub columns tested at FU the average difference is 13%. Owing to the conservative assessment of the concrete in calculating the resistance of the composite section, AISC severely under-predicts the failure loads of the stub column specimens with an average difference of 24%.

The proposed analytical model is also calibrated against the current test data as well as those reported by other researchers. For the specimens tested in the current study, results show that the model gives safe and close estimates of the ultimate capacities with an average difference of 3%. For the specimens tested in previous research projects, the model also closely predicts the failure loads by 3%. Since the specimens cover a wide range of test variables, it is concluded that the analytical model can be used confidently for the design of both normal strength and high-strength rectangular CFSHS columns subjected to eccentric loading.

Table 8.1 Comparison of results for stub columns tested at FU

No.	$B \times H \times t$ (mm)	f_y (MPa)	f'_c (MPa)	λ	e (mm)	N_e (kN)	N_e/N_c			
							EC4	ACI	AISC	Proposed model
S1	120×120×4.0	495	60	10	15	1294	1.01	1.13	1.22	1.04
S2	120×120×4.0	495	60	10	25	1125	1.01	1.15	1.28	1.05
S3	120×120×4.0	495	60	10	30	949	0.91	1.05	1.17	0.96
S4	150×100×4.0	495	60	10	45	810	0.92	1.12	1.24	1.00
S5	150×100×4.0	495	60	10	15	1422	1.02	1.13	1.22	1.04
S6	150×100×4.0	495	60	10	30	1190	1.03	1.17	1.30	1.07
S7	150×100×4.0	495	60	10	45	964	0.97	1.16	1.28	1.04
S8	150×100×4.0	495	60	10	60	763	0.88	1.09	1.20	0.96
S9	180×90×4.0	495	60	10	20	1491	1.02	1.13	1.23	1.09
S10	180×90×4.0	495	60	10	30	1319	1.00	1.13	1.25	1.09
S11	180×90×4.0	495	60	10	40	1208	1.01	1.16	1.30	1.12
S12	180×90×4.0	495	60	10	50	1051	0.96	1.14	1.26	1.08
S13	130×130×4.0	495	60	10	15	1472	0.99	1.12	1.21	1.01
S14	130×130×4.0	495	60	10	25	1305	1.01	1.15	1.29	1.05
S15	130×130×4.0	495	60	10	40	1022	0.94	1.12	1.26	1.01
S16	130×130×4.0	495	60	10	55	789	0.84	1.05	1.17	0.93
Average							0.97	1.13	1.24	1.03
SD							0.06	0.04	0.04	0.05
COV							0.06	0.03	0.03	0.05

Table 8.2 Comparison of results for slender columns tested at FU

No.	$B \times H \times t$ (mm)	λ	e (mm)	N_e (kN)	N_e/N_c				k		
					EC4	ACI	AISC	Propo- sed model	EC4	ACI	AISC
L1	150×100×4.0	60	15	1130	0.97	1.21	1.28	1.02	1.99	2.44	1.46
L2	150×100×4.0	60	30	884	0.95	1.21	1.25	1.02	1.72	1.86	1.34
L3	150×100×4.0	60	45	711	0.90	1.17	1.20	1.00	1.58	1.63	1.27
L4	150×100×4.0	60	60	617	0.89	1.18	1.20	1.01	1.50	1.50	1.22
Average					0.93	1.19	1.23	1.01			
SD					0.04	0.02	0.04	0.01			
COV					0.04	0.02	0.03	0.01			

Notes: $f_y = 495$ MPa; $f'_c = 60$ MPa.

Table 8.3 Comparison of results for columns tested at HIT

No.	$B \times H \times t$ (mm)	f_y (MPa)	f_c' (MPa)	λ	e (mm)	N_e (kN)	N_e/N_c			
							EC4	ACI	AISC	Proposed model
E01	150×150×4.2	550	61	20	30	1678	1.01	1.15	1.27	1.04
E02	150×150×4.2	550	72	20	30	1850	1.03	1.18	1.34	1.08
E03	150×150×4.2	550	61	50	30	1330	0.89	1.11	1.18	0.95
E04	150×150×4.2	550	72	50	60	1020	0.92	1.15	1.21	0.99
E05	180×120×4.2	550	61	20	30	1950	1.15	1.30	1.42	1.17
E06	180×120×4.2	550	72	20	70	1140	0.91	1.11	1.22	0.97
E07	120×80×4.2	550	61	50	20	660	0.81	0.95	0.99	0.83
E08	120×80×4.2	550	72	50	20	855	0.99	1.19	1.24	1.04
E09	200×100×4.2	550	61	20	60	1310	1.03	1.23	1.31	1.12
E10	200×100×4.2	550	72	20	40	1800	1.10	1.27	1.41	1.20
E11	160×80×4.2	550	61	50	60	670	0.93	1.19	1.18	1.05
E12	160×80×4.2	550	72	50	30	1020	0.98	1.23	1.27	1.08
Average							0.98	1.17	1.25	1.04
SD							0.09	0.09	0.12	0.10
COV							0.10	0.08	0.09	0.10

Table 8.4 Comparison of proposed model with previous test data

	No.	$B \times H \times t$ (mm)	L (mm)	f_y (MPa)	f_c' (MPa)	e (mm)	N_e (kN)	M_e (kN)	M_c (kNm)	M_e/M_c
Han et al (2001)	1	120×120×3.84	2600	330	22.3	15	588	8.8	8.4	1.06
	2	120×120×3.84	2600	330	22.3	30	451	13.5	12.9	1.05
	3	120×120×3.84	2600	330	30.0	40	421	16.8	16.1	1.04
	4	120×120×3.84	2600	330	22.3	50	333	16.7	16.7	1.00
	5	120×120×3.84	2600	330	30.0	40	418	16.7	16.2	1.03
	6	120×120×3.84	2600	330	43.0	50	423	21.2	19.8	1.07
	7	140×140×3.84	2560	330	27.7	15	833	12.5	12.3	1.02
	8	140×140×3.84	2560	330	27.7	40	615	24.6	22.6	1.09
	9	140×140×3.84	2560	330	27.7	60	510	30.6	27.3	1.12
	10	140×140×3.84	2560	330	30.0	40	559	22.4	23.2	0.97
	11	140×140×3.84	2560	330	43.0	60	539	32.3	31.6	1.02
	12	120×120×5.86	2600	321	27.7	15	754	11.3	11.4	1.00
	13	120×120×5.86	2600	321	27.7	30	549	16.5	17.6	0.94
	14	120×120×5.86	2600	321	27.7	50	511	25.6	22.9	1.12
	15	140×140×5.86	2560	321	22.6	15	1013	15.2	14.7	1.03
	16	140×140×5.86	2560	321	22.6	30	804	24.1	23.5	1.03
	17	140×140×5.86	2560	321	22.1	40	733	29.3	27.5	1.07
	18	140×140×5.86	2560	321	22.1	60	556	33.4	33.6	0.99
	19	200×200×5.86	2560	321	31.5	30	1793	53.8	52.4	1.03
	20	200×200×5.86	2560	321	26.8	50	1426	71.3	67.6	1.05
	21	200×200×5.86	2560	321	31.5	80	1201	96.1	90.2	1.06
Uy (2001)	HSS3	110×110×5	330	750	27.0	15	1555	23.3	20.8	1.12
	HSS4	110×110×5	330	750	27.0	30	1281	38.4	33.2	1.16
	HSS10	160×160×5	480	750	27.0	25	2024	50.6	52.3	0.97
	HSS11	160×160×5	480	750	27.0	50	1979	99.0	81.7	1.21
	HSS16	210×210×5	630	750	28.8	25	3106	77.7	79.3	0.98
	HSS17	210×210×5	630	750	28.8	50	2617	130.9	128.7	1.02
Han and Yao (2003)	13	130×195×2.65	780	340	18.2	14	872	12.2	10.9	1.12
	14	130×195×2.65	780	340	18.2	14	812	11.4	10.9	1.05
	17	130×195×2.65	780	340	18.2	31	646	20.0	18.8	1.07
	18	130×195×2.65	780	340	18.2	31	610	18.9	18.8	1.01
	25	130×195×2.65	2340	340	18.2	14	670	9.4	9.9	0.95
	26	130×195×2.65	2340	340	18.2	14	635	8.9	9.9	0.90
Varma et al (2002) *	1	305×305×8.6	1500	269	110	N/A	2520	606	532	1.14
	2	305×305×8.6	1500	269	110	N/A	5035	543	520	1.04
	3	305×305×8.9	1500	600	110	N/A	3050	933	915	1.02
	4	305×305×8.9	1500	600	110	N/A	6100	806	788	1.02
	5	305×305×5.8	1500	471	110	N/A	2360	597	594	1.01
	6	305×305×5.8	1500	471	110	N/A	2520	629	599	1.05
	7	305×305×6.1	1500	660	110	N/A	2740	700	763	0.92
	8	305×305×6.1	1500	660	110	N/A	5480	574	682	0.84

Table 8.4 Comparison of proposed model with previous test data (cont'd)

No.	$B \times H \times t$ (mm)	L (mm)	f_y (MPa)	f_c' (MPa)	e (mm)	N_e (kN)	M_e (kN)	M_c (kNm)	M_e / M_c	
1	149×149×4.38	450	262	37.0	200	267	53.9	46.4	1.15	
2	148×148×4.38	450	262	37.0	45	823	38.9	33.6	1.10	
3	215×215×4.38	645	262	22.8	200	503	101.4	94.8	1.06	
4	215×215×4.38	645	262	22.8	60	1141	69.0	64.3	1.07	
5	215×215×4.38	645	262	37.0	200	580	115.4	109.6	1.06	
6	215×215×4.38	645	262	37.0	100	1028	103.2	102.7	1.00	
7	215×215×4.38	645	262	37.0	60	1369	83.0	82.7	0.99	
Fujimo- to et al	8	215×215×4.38	645	262	72.2	1448	146.3	140.9	1.03	
(2004)	9	215×215×4.38	645	262	72.2	60	2014	121.4	121.5	0.99
	10	323×323×4.38	960	262	37.0	200	1479	296.8	292.4	1.01
	11	323×323×4.38	960	262	37.0	60	3306	200.9	192.2	1.03
	12	144×144×6.36	435	618	37.0	200	611	124.0	115.8	1.06
	13	144×144×6.36	435	618	37.0	45	1701	83.7	71.0	1.08
	14	210×210×6.36	630	618	22.8	60	2393	145.7	141.4	1.02
	15	210×210×6.36	630	618	37.0	100	2093	214.2	204.6	1.02
	16	209×209×6.36	630	618	37.0	60	2694	164.3	157.6	1.03
	17	210×210×6.36	630	618	72.2	200	1486	299.4	291.2	1.02
	18	210×210×6.36	630	618	72.2	60	3396	206.3	195.5	1.04
	19	319×319×6.36	960	618	37.0	100	4045	407.7	429.4	0.94
Mean										1.03
SD										0.06
COV										0.06

*Note: The specimens in Varma et al (2002) were tested under constant axial force combined with increasing bending moment.

Fig. 8.1 Comparison of interaction diagrams (EC4, ACI and AISC)

(a) Point A

(b) Point B

(c) Point C

(d) Point D

Fig. 8.2 Stress blocks according to simplified EC4 approach

Fig. 8.3 Constitutive relationship for confined concrete

(a) Deflection curve

(b) Section discretization

Fig. 8.4 Sectional analysis of CFSHS columns under eccentric loading

Fig. 8.5 Flow chart of the computer programme for eccentrically loaded columns

(a) Specimen L1

(b) Specimen L2

Fig. 8.6 Calibration of proposed model against test results (FU)

(c) Specimen L3

(d) Specimen L4

Fig. 8.6 Calibration of proposed model against test results (FU) (cont'd)

Fig. 8.7 Calibration of proposed model against test results (HIT)

Fig. 8.7 Calibration of proposed model against test results (HIT) (cont'd)

Fig. 8.7 Calibration of proposed model against test results (HIT) (cont'd)

Fig. 8.7 Calibration of proposed model against test results (HIT) (cont'd)

(a) Specimens S1-S4

(b) Specimens S5-S8

Fig. 8.8 Calibration of design codes against present test data

(c) Specimens S9-S12

(d) Specimens S13-S16

Fig. 8.8 Calibration of design codes against present test data (cont'd)

Fig. 8.9 Calibration of proposed model against present test data

Fig. 8.10 Calibration of proposed model against previous test data

CHAPTER 9

DESIGN RECOMMENDATIONS FOR HIGH-STRENGTH RECTANGULAR CFSHS COLUMNS

9.1 Columns subjected to axial loading

The resistance of a high-strength rectangular CFSHS column to axial loading can be evaluated based on the resistance of the composite section and resistance reduction factor due to the slenderness effect. Equation (9.1) is proposed to determine the axial load capacity of the rectangular CFSHS column.

$$N_{cr} = \chi N_u \quad (9.1)$$

in which, N_{cr} is the critical load of the column, N_u denotes the resistance of the cross-section, χ is the resistance reduction factor due to slenderness effect. The resistance of the cross-section to axial load and the resistance reduction factor can be determined as follow.

9.1.1 Resistance of a section to axial loading

The resistance of a composite section to axial loading is determined based on that pre-mature local buckling of the steel section does not occur so that the composite section can fully develop its material strengths at the ultimate capacity. Since the allowable H/t limits specified in the design codes, such as EC4 and AISC are too conservative, Equation (9.2) can be referenced for checking the plate slenderness of high-strength rectangular CFSHS columns.

$$H/t = \sqrt{9.31\alpha_r^2 E_s / f_y} \quad (9.2)$$

in which, $\alpha_r = 0.65$ can be assumed for rectangular CFSHS columns (Bradford et al. 1987).

If Equation (9.2) is not satisfied, the following equation applies to consider the effect of local buckling on the ultimate capacity of CFSHS columns.

$$H_e / H = \alpha_r \sqrt{\sigma_b / f_y} \quad (9.3)$$

If pre-mature local buckling of the steel hollow section does not occur, the resistance of the composite section to axial loading can be calculated as

$$N_u = A_s f_y + A_c (0.85 f'_c \eta) \quad (9.4)$$

in which,

$$\eta = 1 + \frac{[1 - 0.65(\beta - 1)^3] (0.37 \xi^{0.82} - 0.19 \xi)}{1 + 0.05(f'_c / 50)^{5.65}} \quad (9.5)$$

$$\beta = H / B \quad (9.6)$$

$$\xi = \frac{A_s f_y}{0.85 A_c f'_c} \quad (9.7)$$

9.1.2 Resistance of a column to axial loading

The effect of slenderness on the resistance of a column can be considered by applying a reduction factor to the plastic resistance of the composite section or by performing comprehensive second-order analysis. When evaluating the behaviour of axially loaded slender columns, the geometric imperfection needs to be taken into account. In the current study, the axial load capacity of slender composite columns is evaluated by assuming the initial load eccentricity to be $0.001L$ to account for the geometric imperfection of the columns. The failure load of slender columns under axial loading is thus determined in the same way as for eccentrically loaded columns. Therefore, the analytical model proposed in Chapter 8 can be used for calculating the critical load of axially loaded slender CFSHS columns.

Since no slender columns are tested under axial loading in the current study, the test data reported by other researchers (Nakamura 1994, Matsui et al. 1995, Han 2000, Han and Yao 2003) are referenced to calibrate this analytical approach. A total of 32 columns were tested in the four research projects. Table 9.1 summarises the material properties and slenderness ratios of the specimens, as well as the

comparison between the tests and the model. The test variables of the specimens cover wide ranges with $f_y=321\sim431$ MPa, $f'_c=18\sim43$ MPa and $\lambda=14\sim103$. Results suggest that the model closely predicts the failure loads of the specimens with a slight un-conservative difference of 1%. The minimum and maximum N_e/N_c ratios are 0.91 and 1.07, respectively. A conclusion can be drawn that the analytical model can be used with confidence to predict the ultimate capacity of axially loaded rectangular CFSHS slender columns.

Although the model can closely predict the axial load capacity of slender composite columns, it is not convenient for practical design, since a comprehensive analysis using computer programmes has to be performed. In order to propose a simplified design approach, a parametric study is performed on 600 composite columns using the analytical model. The parameters of the columns are $f_y=235\sim800$ MPa, $f'_c=30\sim90$ MPa, $\alpha=0.05\sim0.20$ and $\lambda=10\sim100$. The cross-section is assumed to be 150×150 mm. Resistance reduction factors of the columns are obtained by running the computer programme to calculate the critical loads and using the following equation to get the reduction factors.

$$\chi = N_{cr} / N_u \quad (9.8)$$

Table 9.2 summarises the resistance factors of the composite columns determined in the parametric study. Therefore, for a given rectangular composite column, the resistance reduction factor can be determined from this table. Linear interpolation may apply for columns with parameters falling between the figures in the table.

9.2 Columns subjected to pure bending

As presented in Chapter 6, the EC4 approach is more accurate than ACI and AISC to evaluate the flexural strength of high-strength rectangular CFSHS columns. It is thus recommended for the design of high-strength rectangular CFSHS columns subjected to pure bending. Plastic stress distribution in the cross-section can be assumed in the calculation of the flexural strength. Equations (9.8) and (9.9) are recommended for estimating the initial and the serviceability-level flexural

stiffnesses of high-strength rectangular CFSHS columns.

$$EI = E_s I_s + 0.6 E_c I_c \quad (9.9)$$

$$EI = E_s I_s + 0.2 E_c I_c \quad (9.10)$$

Furthermore, following recommendations are made for predicting the flexural stiffness of high-strength rectangular CFSHS columns in a structure:

- (a) Equation (9.9) can be used for evaluating the flexural stiffness if the bending moment acting on the column is lower than 0.20 times the flexural strength of the composite column;
- (b) If the bending moment is higher than 0.20 but lower than 0.60 times the flexural strength of the composite section, Equation (9.10) is recommended for analysis;
- (c) If the bending moment level is higher than 0.60, comprehensive real-time analysis using computer programmes is suggested.

It should be noted that the bending moment level of composite columns in the construction industry, such as building structures, is very low. The columns are subjected to predominant axial loads, which come from the live load and dead load from all stories above the columns. The bending moment, however, only comes from the beams and the floor of the single level on which the columns are located. Therefore, the bending moment level of composite columns in this situation is very low compared with the axial load acting on the columns. Thus, Equations (9.9) and (9.10) can be used for composite columns in most cases.

9.3 Columns subjected to eccentric loading

The ultimate capacity of high-strength rectangular CFSHS columns subjected to eccentric loading can be determined using the analytical model presented in Chapter 8. The constitutive relationships for steel and concrete proposed in Chapter 8 are recommended for analysis. A computer programme is required to calculate the ultimate capacity of CFSHS columns subjected to eccentric loading. Apart from the ultimate capacity, the non-linear load–deflection response of the columns can also be predicted by the proposed analytical model. Since the analytical model incorporates the effects of material strengths, load eccentricity and slenderness on

the behaviour of columns, it can be used with confidence for the design of high-strength rectangular CFSHS columns subjected to eccentric loading.

9.4 Concluding remarks

The recommendations for the design of high-strength rectangular CFSHS columns subjected to axial loading, pure bending and combined axial loading and bending moment have been presented. The determination of axial load capacity for composite columns includes calculating the resistance of the composite section and applying a resistance reduction factor to incorporate the slenderness effect. An equation is proposed for determining the resistance of composite sections to axial loads. A parametric study is conducted on 600 composite columns with various variables. A table of reduction factors is developed for the design of axially loaded rectangular CFSHS columns.

For the determination of flexural strength of high-strength rectangular CFSHS columns, the EC4 approach is recommended. The formula specified in EC4 is recommended for evaluating the initial flexural stiffness. On the other hand, the formula specified in ACI is suggested for predicting the serviceability-level flexural stiffness of high-strength rectangular CFSHS columns.

The determination of ultimate capacity of high-strength rectangular CFSHS columns subjected to eccentric loading is also proposed. An analytical model developed in the study is recommended for calculating the ultimate capacity. Since the model incorporates the confinement of concrete by the steel section and the second-order effect, it can be used with confidence for the design of high-strength rectangular CFSHS columns subjected to axial loading combined with bending moment.

Table 9.1 Comparison of axial loading tests and proposed model

	No.	$B \times H \times t$ (mm)	L (mm)	λ	f_y (MPa)	f'_c (MPa)	e (mm)	N_e (kN)	N_e (kN)	N_e/N_c
Naka- mura (1994)	1	60×60×1.5	500	29	431	23.0	0	227	220	1.03
	2	60×60×1.5	750	43	431	23.0	0	217	215	1.01
	3	60×60×1.5	1000	57	431	23.0	0	192	205	0.94
	4	60×60×1.5	1250	72	431	23.0	0	193	185	1.04
	5	60×60×1.5	1500	86	431	21.6	0	157	159	0.99
	6	60×60×1.5	1750	101	431	21.6	0	146	136	1.07
Matsui et al. (1995)	7	150×150×4.3	600	14	412	28.8	0	1598	1627	0.98
	8	150×150×4.3	1200	28	412	28.7	0	1586	1604	0.99
	9	150×150×4.3	1800	41	412	29.5	0	1573	1588	0.99
	10	150×150×4.3	2700	62	412	27.3	0	1356	1429	0.95
	11	150×150×4.3	3600	83	412	29.0	0	1143	1230	0.93
	12	150×150×4.3	4500	103	412	29.0	0	909	987	0.92
Han (2000)	13	120×120×5.86	2600	75	321	30.0	0	1000	1073	0.93
	14	120×120×5.86	2560	45	321	24.7	0	2083	2291	0.91
	15	120×120×3.84	2600	75	330	26.2	0	753	795	0.95
	16	120×120×3.84	2600	75	330	26.2	0	833	795	1.05
	17	120×120×3.84	2600	75	330	43.0	0	980	973	1.01
	18	140×140×3.84	2560	63	330	26.1	0	1049	1052	1.00
	19	140×140×3.84	2560	63	330	26.1	0	1127	1052	1.07
	20	140×140×3.84	2560	63	330	43.0	0	1323	1297	1.02
Han and Yao (2003)	21	130×130×2.65	780	21	340	18.2	0	760	720	1.06
	22	130×130×2.65	780	21	340	18.2	0	770	720	1.07
	23	240×360×2.65	1440	21	340	18.2	0	2300	2257	1.02
	24	240×360×2.65	1440	21	340	18.2	0	2250	2257	1.00
	25	130×195×2.65	780	21	340	18.2	0	980	959	1.02
	26	130×195×2.65	780	21	340	18.2	0	960	959	1.00
	27	130×195×2.65	2340	62	340	18.2	0	890	900	0.99
	28	130×195×2.65	2340	62	340	18.2	0	815	900	0.91
	29	90×135×2.65	540	21	340	18.2	0	580	590	0.98
	30	90×135×2.65	540	21	340	18.2	0	592	590	1.00
	31	120×240×2.65	720	21	340	18.2	0	1140	1083	1.05
	32	120×240×2.65	720	21	340	18.2	0	1032	1083	0.95
Mean										0.99
SD										0.05
COV										0.05

Table 9.2 Resistance reduction factors for axially loaded CFSHS columns

f_c' (MPa)	f_y (MPa)	α	λ									
			10	20	30	40	50	60	70	80	90	100
30	235	0.05	0.993	0.985	0.973	0.957	0.938	0.916	0.888	0.852	0.804	0.750
		0.10	0.994	0.985	0.974	0.961	0.943	0.922	0.897	0.864	0.820	0.770
		0.15	0.994	0.986	0.976	0.963	0.946	0.926	0.902	0.869	0.829	0.777
		0.20	0.993	0.985	0.976	0.963	0.947	0.928	0.904	0.873	0.834	0.783
	355	0.05	0.993	0.984	0.971	0.949	0.911	0.872	0.828	0.774	0.712	0.656
		0.10	0.993	0.985	0.973	0.957	0.932	0.893	0.842	0.791	0.729	0.663
		0.15	0.993	0.985	0.974	0.959	0.937	0.905	0.854	0.803	0.741	0.672
		0.20	0.993	0.985	0.974	0.959	0.938	0.908	0.864	0.809	0.744	0.675
	450	0.05	0.921	0.920	0.915	0.900	0.866	0.816	0.767	0.709	0.648	0.594
		0.10	0.968	0.961	0.951	0.937	0.912	0.866	0.796	0.733	0.661	0.588
		0.15	0.983	0.975	0.964	0.949	0.926	0.886	0.816	0.746	0.671	0.595
		0.20	0.988	0.980	0.970	0.954	0.931	0.892	0.829	0.751	0.676	0.597
	600	0.05	0.869	0.863	0.853	0.829	0.786	0.737	0.678	0.624	0.569	0.523
		0.10	0.940	0.932	0.920	0.901	0.863	0.799	0.718	0.641	0.563	0.493
		0.15	0.961	0.954	0.941	0.923	0.892	0.834	0.741	0.654	0.572	0.493
		0.20	0.972	0.963	0.950	0.932	0.900	0.846	0.756	0.661	0.577	0.494
	800	0.05	0.855	0.848	0.835	0.806	0.730	0.663	0.592	0.540	0.491	0.451
		0.10	0.930	0.921	0.907	0.884	0.833	0.726	0.633	0.547	0.466	0.408
		0.15	0.953	0.944	0.930	0.907	0.862	0.763	0.650	0.558	0.469	0.398
		0.20	0.963	0.954	0.940	0.917	0.873	0.782	0.660	0.563	0.471	0.394
60	235	0.05	0.993	0.980	0.964	0.942	0.911	0.876	0.826	0.764	0.691	0.614
		0.10	0.991	0.978	0.960	0.938	0.909	0.872	0.825	0.767	0.700	0.628
		0.15	0.991	0.977	0.957	0.934	0.905	0.868	0.822	0.767	0.703	0.637
		0.20	0.991	0.976	0.956	0.932	0.902	0.865	0.820	0.765	0.704	0.646
	355	0.05	0.994	0.983	0.969	0.950	0.925	0.888	0.836	0.771	0.690	0.606
		0.10	0.994	0.984	0.971	0.954	0.931	0.898	0.844	0.789	0.710	0.626
		0.15	0.993	0.985	0.972	0.956	0.934	0.905	0.860	0.800	0.721	0.637
		0.20	0.994	0.985	0.973	0.957	0.935	0.907	0.864	0.803	0.726	0.640
	450	0.05	0.934	0.983	0.964	0.934	0.900	0.853	0.790	0.723	0.646	0.567
		0.10	0.993	0.983	0.969	0.944	0.909	0.868	0.806	0.731	0.647	0.566
		0.15	0.993	0.984	0.971	0.950	0.917	0.872	0.819	0.742	0.655	0.570
		0.20	0.993	0.984	0.971	0.952	0.921	0.881	0.826	0.751	0.662	0.574
	600	0.05	0.991	0.980	0.960	0.911	0.858	0.794	0.722	0.659	0.587	0.514
		0.10	0.993	0.983	0.967	0.938	0.879	0.812	0.732	0.643	0.564	0.493
		0.15	0.993	0.983	0.969	0.944	0.898	0.824	0.743	0.653	0.560	0.483
		0.20	0.993	0.983	0.969	0.946	0.906	0.832	0.751	0.657	0.562	0.479
	800	0.05	0.898	0.891	0.877	0.835	0.781	0.716	0.650	0.592	0.525	0.459
		0.10	0.949	0.940	0.926	0.895	0.817	0.733	0.640	0.552	0.484	0.422
		0.15	0.969	0.960	0.945	0.917	0.857	0.749	0.651	0.552	0.468	0.403
		0.20	0.978	0.969	0.953	0.927	0.870	0.761	0.658	0.555	0.464	0.394

Table 9.2 Resistance reduction factors for axially loaded CFSHS columns (cont'd)

f_c' (MPa)	f_y (MPa)	α	λ									
			10	20	30	40	50	60	70	80	90	100
90	235	0.05	0.992	0.980	0.960	0.930	0.882	0.826	0.755	0.674	0.589	0.510
		0.10	0.991	0.978	0.956	0.919	0.872	0.819	0.753	0.680	0.602	0.529
		0.15	0.991	0.976	0.951	0.909	0.867	0.815	0.753	0.683	0.611	0.544
		0.20	0.990	0.974	0.946	0.906	0.864	0.813	0.753	0.686	0.616	0.562
	355	0.05	0.993	0.982	0.967	0.944	0.911	0.863	0.794	0.706	0.611	0.524
		0.10	0.992	0.981	0.966	0.945	0.913	0.869	0.807	0.726	0.637	0.552
		0.15	0.992	0.981	0.965	0.944	0.915	0.872	0.814	0.737	0.654	0.569
		0.20	0.992	0.980	0.964	0.943	0.914	0.873	0.816	0.743	0.661	0.582
	450	0.05	0.993	0.981	0.965	0.943	0.909	0.859	0.786	0.694	0.598	0.511
		0.10	0.993	0.983	0.967	0.947	0.918	0.872	0.805	0.716	0.618	0.528
		0.15	0.993	0.983	0.969	0.949	0.922	0.878	0.815	0.730	0.632	0.540
		0.20	0.993	0.984	0.970	0.951	0.924	0.885	0.819	0.735	0.639	0.546
	600	0.05	0.993	0.980	0.952	0.916	0.867	0.801	0.731	0.645	0.555	0.473
		0.10	0.993	0.982	0.964	0.928	0.878	0.815	0.727	0.639	0.551	0.470
		0.15	0.993	0.983	0.967	0.938	0.888	0.823	0.741	0.642	0.550	0.468
		0.20	0.993	0.983	0.968	0.942	0.895	0.830	0.748	0.648	0.550	0.467
	800	0.05	0.973	0.962	0.936	0.877	0.810	0.735	0.670	0.590	0.507	0.432
		0.10	0.981	0.974	0.957	0.914	0.826	0.740	0.640	0.560	0.482	0.410
		0.15	0.990	0.979	0.962	0.928	0.842	0.750	0.648	0.547	0.467	0.397
		0.20	0.992	0.981	0.964	0.933	0.863	0.756	0.653	0.547	0.460	0.389

CHAPTER 10

CONCLUSIONS

10.1 Conclusions

The overall objectives of this study were to investigate the behaviour of high-strength rectangular CFSHS columns and to propose design recommendations. A total of 117 specimens were tested in the research programme. Analytical models are proposed for the numerical analyses of high-strength rectangular CFSHS columns subjected to concentric loading, pure bending and eccentric loading. The test data obtained in the current study as well as those reported by other researchers are employed to calibrate the proposed analytical models. Based on the experimental and analytical studies as presented in the previous chapters, the conclusions are drawn as follow.

- (1) For stub columns subjected to concentric loading, the strength of the concrete is improved due to the confinement effect. The strength increase is dependent on the cross-sectional aspect ratio, material strengths and the volumetric steel-to-concrete ratio. Besides, the ductility performance of the concrete is found to be enhanced. The inherent brittle crushing of the high-strength concrete can be avoided if confined by rectangular steel hollow sections.
- (2) The resistance of composite sections to axial loads can be determined using the analytical model developed in the current study.
- (3) The axial load capacity of slender columns can be calculated by applying a reduction factor to the resistance of the composite section. The reduction factor can be obtained either using the computer programme or the table developed in the study.
- (4) The flexural strength of columns subjected to pure bending can be determined either using EC4 or the proposed analytical model. The formulas in EC4 and

ACI are recommended for evaluating the initial and serviceability-level flexural stiffnesses, respectively.

- (5) For columns subjected to eccentric loading, the position of local buckling was affected by the slenderness and the load eccentricity ratio. For stub and intermediate (slenderness, $\lambda = 20$) columns with $e/H = 0.10 \sim 0.40$, local buckling occurred at approximately $0.25L$. For stub and intermediate columns with $e/H > 0.40$, local buckling occurred near the ends. For slender columns with $\lambda = 50$ and 60 , overall buckling was observed.
- (6) For columns subjected to eccentric loading, the analytical model developed in the study can be used to predict the ultimate capacity. The model incorporates the strength increase of the concrete and the slenderness effect but ignores the slip between the steel and the concrete. The calibration of the model with current test data as well as those reported by other researchers shows that it can closely estimate the ultimate capacity of high-strength rectangular CFSHS columns.

10.2 Recommendations for future research

Although an extensive investigation of the behaviour of high-strength rectangular CFSHS columns has been conducted in the current research programme, there is still a need for the following to be examined in order to achieve a better understanding of the behaviour and to develop design formulas for high-strength rectangular CFSHS columns.

(1) Size effect

It is well-known that the strength of concrete depends on the size of the member or sample (cube or cylinder) used in the test. Due to the capacity limit of the testing machine, which is normally 5,000 kN, composite column specimens are usually designed to be smaller than 150×150 mm. The strength of the concrete is determined by testing concrete cubes (150×150×150 mm) or cylinders (150×300 mm). There is uncertainty whether the strength of the concrete samples can accurately represent the strength of the concrete in composite column specimens. Moreover, composite columns used in the construction industry usually have cross-sections larger than 600×600 mm. The

implementation of formulas derived based on test data of small-size specimens may result in un-conservative estimates of the ultimate capacity for columns with large dimensions. Extensive studies on the size effect are to be carried out so that the proposed formulas can be used with confidence.

(2) Creep and shrinkage

Creep and shrinkage are two inherent properties of concrete. Shrinkage can contribute to failure of bond between the steel and the concrete in composite columns. Previous studies are mainly concerned with normal strength concrete mixed in conventional method. With the increasingly popular use of high-performance concrete, the effects of creep and shrinkage on the behaviour of high-performance composite columns should be studied. High-performance concrete usually has a lower water-to-cement ratio than normal strength concrete. Extensive studies are expected to investigate the effects of material properties, geometric parameters and sustained load level on the behaviour of high-performance CFSHS columns.

(3) Seismic performance

The seismic requirement is a critical factor to consider for the design of composite columns in earthquake sensitive regions. High-strength rectangular CFSHS columns are expected to be less ductile than normal strength columns. Extensive studies on the seismic performance of high-strength rectangular CFSHS columns and frames are essentially required before they can be widely adopted in structures at high risk of earthquake.

(4) Fire resistance

Fire resistance is a major concern for CFSHS columns since the steel hollow section is exposed. Research in this field encompasses the determination of the temperature field, development of constitutive relationships for steel and concrete and consideration of creep for the steel section. The effects of duration of fire, material strengths, geometric properties and type of loading on the fire-resistant behaviour of high-strength rectangular CFSHS columns are to be extensively investigated.

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APPENDIX

Publications by the author during his PhD candidature

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