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Flexural performance of rectangular CFST members

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ABSTRACT

This paper presents a finite element analysis (FEA) modeling to study the flexural performance of rectangular concrete filled steel tubular (CFST) members with compact, non-compact or slender element sections. Seventy test results are used to verify the FEA modeling. Generally good agreements were achieved in the ultimate bending capacity and the curvilinear trends of the moment versus mid-span deflection relations of the composite member between the experimental results and the FEA results. The FEA modeling is then used to investigate the residual failure patterns of the core concrete, the typical residual deformations of outer steel tube and the stress and strain distributions across the composite section in the whole loading procedure. Analysis results show that interaction of steel and concrete in the composite beam offers stress redistribution in steel and concrete which makes the rectangular CFST beam have high flexural capacity and ductility. Finally, the reliability analysis method is used to calibrate the existing design formulae on composite beam in EC4 (2004), AISC (2010) and DBJ/T13-51-2010 (2010). It was found that all the design formulae achieved adequate reliability index.

1. Introduction

Concrete-filled steel tubes (CFST) with rectangular sections are widely used in many modern structures as columns or piers in buildings or infrastructures. CFST members under pure bending, the extreme case of beam-column in which there is no axial load present, are classified as a compact section, non-compact section or slender element section according to the effect of the local buckling of steel tube on the capacity of CFST beams in AISC [1] and EC4 [2]. Compact sections can form a plastic hinge with the rotation capacity required for plastic design. Non-compact sections can develop the fully plastic moment but have limited rotation capacity or reach the first yield moment, but local buckling prevents the development of the fully plastic moment. Slender element sections cannot reach first yield moment because of local buckling effect. The slender limit for compact section, non-compact section, or slender-element section are summarized in Table 1 for width-to-thickness ratio (B/t) and depth-to-thickness ratio (D/t) of rectangular hollow steel tube (RHS) in the existing design codes [1,3,4] and rectangular CFST in the exiting design codes [1,2,5] where elastic modulus of steel E_s=200,000 MPa in [1] and yield strength of steel f_y in MPa. For rectangular hollow steel tubes (RHS), flange plane is more slender than web plane, so the section type is decided by the width-to-thickness (B/t) according to [3]. It can be seen that the B/t limit of CFST members is nearly 1.5 times to that of the corresponding hollow steel tube. There is little difference in the definitions of the B in width-to-thickness ratio (B/t) in existing codes. In order to unify the expression of B/t, where B is the outer width of the section.

Flexural behavior of rectangular CFST columns have been previously studied by Furlong [6], Tomii and Sakino [7], Lu and Kennedy [8], Uy [9,10], Guo and Liu [11], Han [12] and Han et al. [13]. The depth-to-width ratio (D/B) varied from 1 to 2, the strength of core concrete f_{cu} ranged from 23.53 MPa to 81.3 MPa and yield stress of steel f_y varied from 194 MPa to 750 MPa, as summarized in Table 2. These study results show that (1) the flexural capacity strength of rectangular CFST is about 10–50% over that of bare steel sections; (2) the rectangular CFST members behave in ductile manner; (3) the main failure mode of rectangular CFST include the outward local buckling of out steel tube near the load position and flexural crack of core concrete at the bottom of section in the pure bending region generally; the multiple fold of outer steel tube were observed under large deformation by Zhao et al. [14] and (4) the shear-to-depth ratio (a/D) from 1.03 to 6 has a very moderate influence on the behaviors of the rectangular CFST members.

However there is still a need to develop a finite element analysis (FEA) modeling of such composite beams to further explain these phenomena, e.g., the ductile behavior, the increased

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Nomenclature

a	shear span
A_c	concrete cross-sectional area
A_s	steel cross-sectional area
a/D	shear span to depth ratio
B	width of rectangular steel tube
D	depth of rectangular steel tube
E_c	elastic modulus of concrete
E_s	elastic modulus of steel
f'_c	cylinder strength of concrete
f_{ck}	characteristic concrete strength ($f_{ck}=0.67f_{cu}$ for normal strength concrete)
f_{cu}	characteristic 28-day concrete cube strength
f_y	yield strength of steel
f_u	ultimate strength of steel
M	bending moment
M_u	ultimate bending strength of the composite section
t	wall thickness of steel tube
W_{sc}	section modulus of the rectangular composite beams
α	steel ratio ($\alpha = A_s/A_c$)
σ	stress
ε	strain
ξ	confinement factor ($\xi = (A_s f_y)/(A_c f_{ck})$)
ϕ	curvature

Table 1
Limits of B/t and D/t of the rectangular hollow steel tube and CFST.

Code	Rectangular hollow steel tube (RHS)			Code	Rectangular CFST		
	I (compact)	II	III (non-compact)		I (compact)	II	III (non-compact)
B/t							
GB50017-2003(2003)	$40\sqrt{235/f_y}$	–	–	DBJ/T13-51-2010 (2010)	$60\sqrt{235/f_y}$	–	–
EC3 (2005)	$33\sqrt{235/f_y}$	$38\sqrt{235/f_y}$	$42\sqrt{235/f_y}$	EC4 (2004)	$52\sqrt{235/f_y}$	–	–
AISC(2010)	$1.12\sqrt{E_s/f_y}$	–	$1.4\sqrt{E_s/f_y}$	AISC(2010)	$2.26\sqrt{E_s/f_y}$	–	$3\sqrt{E_s/f_y}$

Code	RHS		
	I (compact)	II	III (non-compact)
D/t			
GB50017-2003(2003)	$80\sqrt{235/f_y}$	–	–
EC3 (2003)	$72\sqrt{235/f_y}$	$83\sqrt{235/f_y}$	$124\sqrt{235/f_y}$
AISC(2010)	$3.76\sqrt{E_s/f_y}$	–	$5.70\sqrt{E_s/f_y}$

moment capacity as well as the multiple fold mechanism. A reliability analysis on the design formulae in the existing design codes based on the large number of experimental data is also necessary.

The main objectives of this paper are thus three folds: first, to develop a FEA modeling for the rectangular CFST members with compact section, non-compact section or slender-element section; second, to analyze the flexural performance and load transfer mechanism of rectangular CFST in a detail way by using the FEA modeling. And third, to check the reliability of the design formulae listed in AISC [1], EC4 [2] and DBJ13-51-2010 [5].

2. Finite element analysis (FEA) modeling

2.1. General description of the FEA modeling

The finite element analysis (FEA) modeling is established using the ABAQUS software [15].

2.1.1. Material constitutive model

A five-stage stress–strain model of steel introduced in [16] is used herein. In this model, the deformation of steel includes elastic, elastic–plastic, plastic, hardening and fracture, as shown in Fig. 1(a), where f_p, f_y and f_u are respectively the proportional limit, yield and ultimate strength of the steel, and $\varepsilon_e=0.8f_y/E_s$, $\varepsilon_y=1.5\varepsilon_e$, $\varepsilon_{uy}=10\varepsilon_y$, $\varepsilon_u=100\varepsilon_y$. The modulus of elasticity E_s and Poisson's ratio of steel are taken as 200,000 N/mm^2 and 0.3, respectively.

The fabrication of rectangular steel tubes includes welding and cold bending which introduces residual stresses at the welding and cold-formed region. The test results of residual stresses on steel plates for welded rectangular steel tubes columns were presented in [17], where they were shown to be about 15–25% of the yielding stress in compression. The average value of 20% of the yielding stress in compression is selected in the analysis in this paper. The schematic diagram of stress–strain relation for cold-formed corner steel as shown in Fig. 1(b), where $E_{s1}=103{,}000$ N/mm^2, $E_{s2}=26{,}000$ N/mm^2, $E_{s3}=10{,}300$ N/mm^2 are the modulus of different stages of cold formed steel as described by Abdel-Rahman [18].

The concrete damaged plasticity model is used to simulate the behavior of core concrete in the rectangular CFST. The modulus of elastic of concrete is taken as $4730\sqrt{f'_c}$, where f'_c is the cylinder compression strength of concrete (in N/mm^2), and Poisson's ratio is taken as 0.2 according to ACI 318 [19]. A stress–strain relation model for concrete under compression presented in [16] is used, in which the increase of plasticity for core concrete due to the passive confinement of the outer steel tube is considered. The fracture energy for tensile concrete given in [15] is applied for the tensile softening behavior of concrete.

A surface-based interaction with a contact pressure model in the normal direction and a Coulomb friction model in the tangential direction is used to simulate the interface between steel tube and core concrete. The friction factor is 0.6 recommended by Han et al. [16].

2.1.2. Element, boundary condition and mesh

Based on the geometric characteristics of concrete and steel in the rectangular CFST, the first order reduced-integration 3D hexahedral