INELASTIC BEHAVIOR OF CONCRETE-FILLED THIN-WALLED STEEL TUBULAR COLUMNS SUBJECTED TO LOCAL BUCKLING

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ABSTRACT: This paper is concerned with the inelastic behavior of axially loaded concrete-filled thin-walled steel tubular columns subjected to local buckling. A nonlinear fiber element analysis program accounting for local buckling effects is developed for predicting the ultimate strength and ductility of concrete-filled thin-walled steel tubular columns. The effects of local buckling are taken into consideration by using the local buckling equations and effective width formulas for steel plates. The fiber element analysis technique is verified by experimental results and is employed to study the effects of steel ratios and concrete strengths on the ultimate strength and ductility of concrete-filled thin-walled steel tubular columns.

KEYWORDS: composite column; ductility; fiber element method; local buckling; strength.

1. INTRODUCTION

High strength structural steels and concrete are increasingly used with thin steel plates in concrete-filled steel tubular (CFST) columns. However, this gives a rise to local buckling. High strength concrete may also reduce the ductility of CFST columns because of its brittle nature. Design codes such as Eurocode 4 [1], LRFD [2] and ACI 318-02 [3] do not consider the effects of the plate local buckling on the ultimate strength of CFST columns. Tests on thin-walled CFST columns showed that thin steel plates might buckle award from the concrete core [4-6]. The post-local buckling behavior of steel plates in composite members was reported by Liang and Uy [7] and Liang et al. [8, 9].

The fiber element method has been presented by El-Tawil et al. [10] for the nonlinear analysis of concrete-encased composite columns under axial load and biaxial bending. El-Tawil and Deierlein [11] studied the ultimate strength and ductility of concrete-encased composite columns. Lakshmi and Shanmugam [12] presented a semi-analytical model for analyzing CFST columns. Liew et al. [13] developed an advanced analysis program for the nonlinear analysis of steel frames with composite beams. The effects of local buckling, however, have not been considered in nonlinear analysis methods for thin-walled CFST columns.

In this paper, the ultimate strength and ductility of short thin-walled CFST columns with local buckling effects are investigated by using a nonlinear fiber element analysis technique. Design formulas for critical local buckling and effective width formulas are employed in the fiber element analysis method to account for local buckling effects. The accuracy of the fiber element analysis method is established by comparisons with experimental results. The fiber element analysis technique is then employed to study the effects of steel ratios and concrete strengths on the strength and ductility of thin-walled CFST columns subjected to local buckling.

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2. FIBER ELEMENT ANALYSIS

2.1 CONSTITUTIVE MODELS FOR STEEL

In the fiber element method, the composite section is discretized into many fiber elements. The uniaxial stress-strain relationships are used for materials. Stress resultants are obtained by numerical integration of stresses through the composite section. The fiber stresses for structural steels with residual stresses are calculated using The Ramberg-Osgood formula [14], which is expressed by

$$\varepsilon_s = \frac{\sigma_s}{E_s}\left[1 + \frac{3}{7}\left(\frac{\sigma_s}{\sigma_{0.7}}\right)^n\right] \tag{1}$$

where σ_s is the longitudinal stress in steel, ε_s is the longitudinal strain in steel, E_s is the Young's modulus of steel, $\sigma_{0.7}$ is the stress corresponding to $E_{0.7} = 0.7E_s$, and n is the knee factor that defines the sharpness of the stress-strain curve. The knee factor n = 25 is used in the fiber element analysis program to account for the isotropic strain hardening of steel sections [7].

2.2 CONSTITUTIVE MODELS FOR CONCRETE

It is assumed that the confinement effect increases only the ductility of the encased concrete in a CFST column but not its ultimate load [15]. The general stress-strain curve for concrete in CFST columns is depicted in Figure 1. The part OA of the stress-strain curve is modeled using the equation suggested by Mander et al. [16] as

$$\sigma_c = \frac{f_c^{'}\gamma\left(\varepsilon_c / \varepsilon_c^{'}\right)}{\gamma - 1 + \left(\varepsilon_c / \varepsilon_c^{'}\right)^{\gamma}} \tag{2}$$

where σ_c is the longitudinal compressive concrete stress, $f_c^{'}$ is the compressive cylinder strength of concrete, ε_c is the longitudinal compressive concrete strain, $\varepsilon_c^{'}$ is the strain at $f_c^{'}$. The parameter γ is determined by

$$\gamma = \frac{E_c}{E_c - \left(f_c^{'} / \varepsilon_c^{'}\right)} \tag{3}$$

where E_c is the Young's modulus of concrete. The strain $\varepsilon_c^{'}$ is taken as 0.002 for concrete strength under 28 MPa and 0.003 for concrete strength over 82 MPa and is determined as a linear function of the concrete strength between 28 and 82 MPa. The parts AB, BC, CD of the stress-strain curve for confined concrete depicted in Figure 3 are defined as follows:

$$\sigma_c = f_c^{'} \text{ for } \varepsilon_c^{'} < \varepsilon_c \le 0.005 \tag{4}$$

$$\sigma_c = \alpha f_c^{'} + 100\left(0.015 - \varepsilon_c\right)\left(f_c^{'} - \alpha f_c^{'}\right) \text{ for } 0.005 < \varepsilon_c \le 0.015 \tag{5}$$

$$\sigma_c = \alpha f_c^{'} \text{ for } \varepsilon_c > 0.015 \tag{6}$$

where α is taken as 1.0 when the width-to-thickness ratio (*B/t*) of the composite column is less than 24 and is taken as 0.0 when the *B/t* ratio is greater than 64 as suggested by Tomii and Sakino [15]. For *B/t* ratios between 24 and 64, α is taken as 0.6 in the fiber element analysis program.

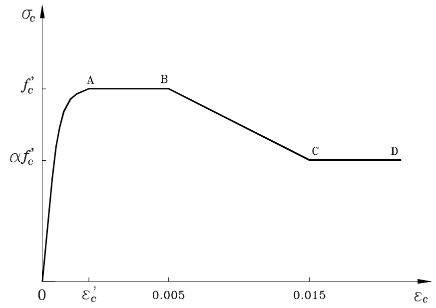


Figure 1. Stress-strain curve for concrete

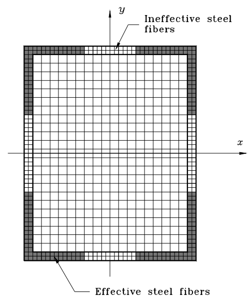


Figure 2. Effective fibers in CFST section

2.3 CRITICAL LOCAL BUCKLING

Local buckling of steel plates depends on the width-to-thickness ratios, stress states, boundary conditions, initial geometric imperfections and residual stresses. For CFST columns under axial compression, the critical local buckling stresses of thin steel plates can be determined by [9]

$$\frac{\sigma_{cb}}{f_y} = 0.5507 + 0.005132\left(\frac{b}{t}\right) - 9.869\times10^{-5}\left(\frac{b}{t}\right)^2 + 1.198\times10^{-7}\left(\frac{b}{t}\right)^3 \tag{7}$$

where σ_{cb} is the critical local buckling stress of the plate with imperfections, b is the width of the plate, t is the thickness of the plate and f_y is the yield strength of steel plates. Equation (7) accounts for the initial out-of-plane deflection of $0.1t$ and residual compressive stress of $0.25f_y$ and can be used for steel plates with *b/t* ratios ranging from 30 to 100.

2.4 POST-LOCAL BUCKLING

The effective width concept is usually used to express the post-local buckling strength of thin steel plates as depicted in Figure 2. An effective width formula proposed by Liang et al. [9] is employed in the fiber element analysis program and it is expressed by

$$\frac{b_e}{b} = 0.5554 + 0.02038\left(\frac{b}{t}\right) - 3.944\times10^{-4}\left(\frac{b}{t}\right)^2 + 1.921\times10^{-6}\left(\frac{b}{t}\right)^3 \tag{8}$$

where b_e is the effective width of a steel plate. The above effective width formula accounts for the initial out-of-plane defection of $0.1t$ and residual compressive stress of $0.25f_y$ and can be used for steel plates with *b/t* ratios ranging from 30 to 100. In the fiber element analysis program, the progressive local and post-local buckling of steel plates in concrete-filled steel box columns is simulated by gradually redistributing the normal stresses within the steel plates.

3. SECTION AND DUCTILITY PERFORMANCE

In the LRFD code [2], a column is classified as composite if it has a structural steel area to the cross-sectional area ratio of more than 0.04 otherwise it is treated as a concrete column. In Eurocode 4 [1], the steel contribution ratio in a composite column section, which is defined as the ratio of the steel section strength to the composite section strength, must be greater than 0.2. To evaluate the section performance of composite columns, a performance index is proposed here as

$$PI_s = \frac{\sum_{i=1}^{ns} \sigma_{u,i} A_{s,i}}{\sum_{i=1}^{ns} \sigma_{u,i} A_{s,i} + \sum_{j=1}^{nc} \sigma_{u,j} A_{c,j}} \tag{9}$$

where $\sigma_{u,i}$ is the longitudinal stress of steel fiber *i* at the ultimate load and $\sigma_{u,j}$ the longitudinal stress of concrete fiber *j* at the ultimate load. The section performance index accounts for the effects of cross-sectional areas and material strengths of steel and concrete and *b/t* ratios.

To evaluate the axial ductility performance of CFST columns, the ductility performance index is defined as

$$PI_d = \frac{\varepsilon_{0.95}}{\varepsilon_y} \tag{10}$$

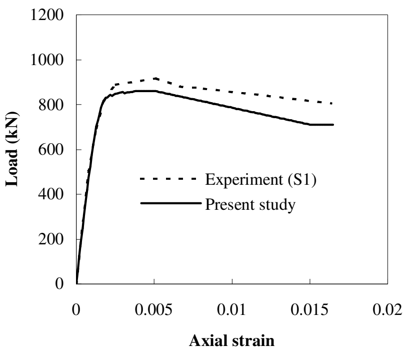
where $\varepsilon_{0.95}$ is the axial strain when the load falls to 95% of the ultimate load and ε_y is the axial strain when the composite section is at yield. The axial strain ε_y is approximately defined as the strain when the load attains 95% of the ultimate load.

4. COMPARISONS WITH EXPERIMENTAL RESULTS

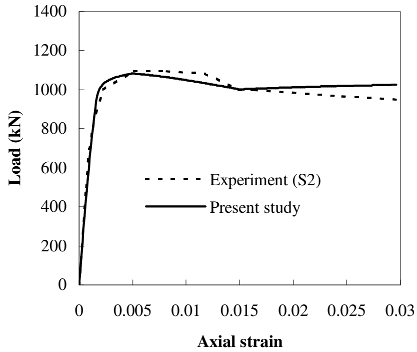
The fiber element analysis results are compared with experimental data presented by Schneider [17] to verify the method. In the present fiber element analysis, the maximum concrete compressive stress in the constitutive model was taken as $0.85 f_c^{'}$ for all specimens. The load-axial strain curves for these concrete-filled steel box columns obtained by the fiber element analysis and experiments are depicted in Figure 3. The figure demonstrates that the fiber element analysis technique predicted very well the axial stiffness, ultimate strengths and post-peak behavior of the test specimens.

5. EFFECTS OF STEEL RATIOS

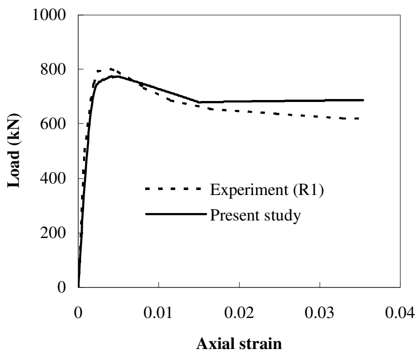
The composite column section (600 × 600 mm) with steel ratios ($\beta = A_s / A_g$) of 0.04, 0.08 and 0.12 was analyzed using the fiber element technique. Material properties were: $f_y = 250$ MPa and $f_c^{'} = 28$ MPa and $E_s = 200$ GPa. The maximum compressive concrete strength was taken as $0.85 f_c^{'}$ in the material model. The load-axial strain curves obtained for CFST columns are depicted in Figure 4(a), where P_0 is the ultimate load of the composite section with a steel ratio of 0.04. The figure shows that increasing the steel ratio increases the ultimate load and axial stiffness of the column. The section performance index increased from 0.2 to 0.44 and 0.55 when the steel ratio increased from 0.04 to 0.08 and 0.12, respectively. The ductility performance index of the section with steel ratios of A_s / A_g = 0.04, 0.08 and 0.12 were 3.97, 5.01 and 5.47, respectively. It is noted that the 0.04 steel ratio of a composite section corresponds to the 0.2 steel contribution ratio (or the section performance index) of the section. Although the section satisfied the minimum requirement of Eurocode 4 [1] on the steel contribution ratio, the steel box wall was very slender. It is suggested that the sections of CFST columns should be designed to have a performance index or the steel contribution ratio as high as 0.5 to achieve high structural performance.



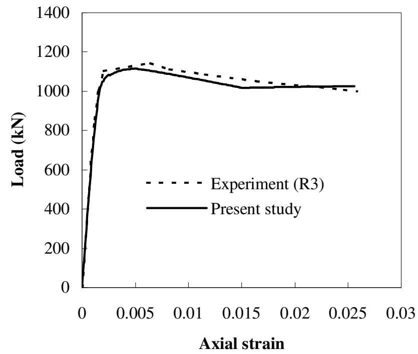
(a) Specimen S1



(b) Specimen S2



(c) Specimen R1



(d) Specimen R3

Figure 3. Comparison of present study with experimental results

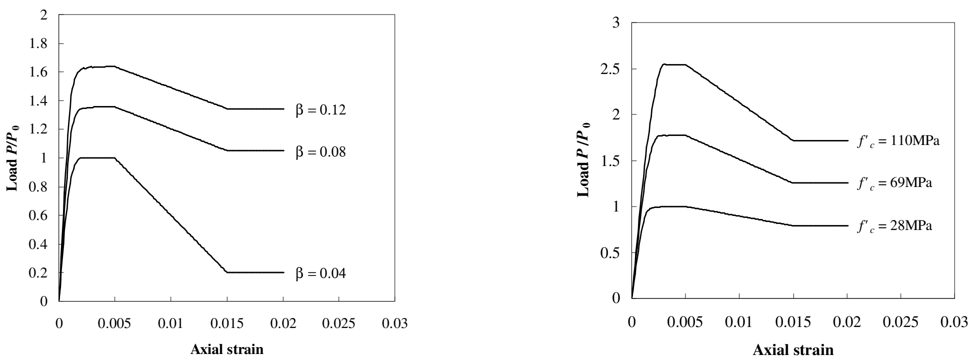


Figure 4. Behavior of CFST columns: (a) effects of steel ratios; (b) effects of concrete strengths

6. EFFECTS OF CONCRETE STRENGTHS

The effects of concrete strengths on the ultimate strength and ductility of CFST columns were investigated using the fiber element method. The dimension of the composite section was 600 × 600 mm with a *B/t* ratio of 50 and the steel box column was filled with 28, 69 and 110 MPa concrete, respectively. Figure 4(b) shows the load-axial strain curves for the column filled with difference strength concrete. It is seen that increasing the concrete strength increases the ultimate load of the column. However, when the concrete strength increased from 28 to110 MPa, the ductility performance index of the composite section decreased from 4.9 to 2.6.

7. CONCLUSIONS

In this paper, the ultimate strength and ductility of short concrete-filled thin-walled steel box columns have been investigated using the fiber element analysis technique. The progressive local and post-local buckling of a thin-walled CFST column is modeled by gradually distributing the normal stresses within the steel box. The effects of steel ratios and concrete strengths on the ultimate strength and ductility of CFST columns were investigated. The 4% limit on the steel ratio or the 0.2 limit on the steel contribution ratio imposed in current design codes leads to the use of very slender steel tube walls in CFST columns. It is suggested that CFST columns should be designed to have a section performance index as high as 0.5 to be considered as efficient composite sections.

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