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ANALYSIS OF SQUARE CONCRETE-FILLED COLD-FORMED STEEL TUBULAR COLUMNS UNDER AXIAL CYCLIC LOADING

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ABSTRACT: This research studied the behavior of the square concrete-filled cold-formed steel tube (CFST) columns under axial cyclic loading. A total of 9 specimens were tested. All tested specimens are 75×75 millimeter square tube section. The parameters varied in the test are the thickness of steel tube, the compressive strength of filled concrete, and specimen length. The values of tube thickness are 1.8 and 3 millimeters. The column lengths are 500 and 1000 millimeters. In-filled concrete compressive strengths are 0 (unfilled), 20 and 40 MPa, respectively. Then the experimental strengths were compared with the design strengths computed from various international design codes. Finally, the specimens were analyzed by finite element software. The analytical results showed fairly close agreement with experimental results in terms of buckling mode, load-deformation response, the tension capacity and the decreasing compression capacity under cyclic load.

Keywords: Concrete filled hollowed square steel column, Axial capacity, Cyclic loading, Finite element analysis

1. INTRODUCTION

For a moderate earthquake-prone area, many structures have been constructed without seismic consideration. However, high important buildings with the high consequent loss, e.g. school buildings, hospital buildings, other crowded public buildings, have been raised for more reliable seismic safety. Therefore, it is imperative to achieve methods of reducing earthquake damage to an economically supportable level. To this end, many countries and owners of buildings are investing into a seismic upgrade, strengthening and retrofitting of their buildings. Among various structural upgrading methods, utilization of bracing member is considered as an effective one. Such member often employs steel rectangular hollow section (RHS) and square hollow section (SHS). The main reasons can be (1) underweight (2) quick installation work (3) aesthetic acceptance. To achieve better seismic performance, overall structural stability must be maintained by which energy will be dissipated through inelastic cyclic deformation of the bracing members. Nevertheless, hollow thin section steel column can be locally buckled leading to unfavorable inelastic behavior.

Static load carrying capacity of concrete-filled steel tube columns (CFST) has been investigated by many researchers [1–3]. Early investigation of inelastic buckling of thin hollow columns under cyclic loading was done [4]. The study concluded the distinct effect of buckling on stiffness and strength degradation. Tearing of columns in cyclic tension was observed at the buckled section. The authors also noted the effect of supports in which the fixed-ended type provided higher initial stiffness and higher buckling load with stable inelastic behavior compared to the pinned-ended type. The slenderness of the steel column is a significant factor affecting compressive load capacity. Hence, some researchers [5 –7] had paid attention to determine the formula for estimating the loading capacity in terms of the slenderness ratio. Goggins [8] conducted an experimental study on the response of bare rectangular and square hollow steel members to monotonic and cyclic axial loadings. Stable hysteresis behavior was maintained until the commencement of local buckling. The slenderness of the column was also noted as a primary factor affecting strength and ductility degradation. Authors [9] also found that concrete in-filled help to reduce the buckling of steel tube. Many researchers [10-11] compared the tested results with strength predicted by various design codes of practice. Niranjan [12] found that the compressive strength of tested results shown higher than the analytical strength results. Some codes had given the most conservative results for confining effect of in-filled steel columns [13].

This work presented the study on the behavior of concrete-filled steel tube column under cyclic load. The studied parameters are the compressive strength of in-filled concrete. Then the test results were compared with ultimate loads estimated from

ACI and AISC code of practices [14-15] and finite element analysis [16]. The result shows better strength and ductility of in-filled concrete column compare with an empty steel column. Hence, the results are useful for a designer who would like to design bracing member for strengthening building which located in the seismic risk zone.

2. MATERIALS AND METHODS

2.1 Experiments

Nine full-scale specimens of 75-millimeter square-hollow steel columns were tested in this study. The main parameters were the in-filled concrete strength and steel column sectional shape and dimension. The average compressive strengths of in-filled concrete are 20 and 40 MPa. Two values of column length are 500 mm and 1000 mm, and thickness of hollow steel column are 1.8 and 3.0 mm. All specimens are coded as shown in table 1. The symbols "A" and "B" represented the column length of 100 cm, and 50 cm, respectively. The number follows the symbol "F" shows the average compressive strength of in-filled concrete. The sample of the tested specimen is shown in Fig. 1. Slenderness ratio of type A specimens are 33.90, type B specimens are 16.95 and 16.78 for 1.8 and 3.0 mm wall thickness, respectively.

Table 1 Specimen name lists

f_c' (MPa)	L = 50 cm t = 1.8 mm	L = 50 cm t = 3 mm	L = 100 cm t = 1.8 mm
empty	B1.8F0	B3.0F0	A1.8F0
20	B1.8F20	B3.0F20	A1.8F20
40	B1.8F40	B3.0F40	A1.8F40

Tested specimens were installed on the universal testing machine (INSTRON 2000 kN) as shown in Fig. 2, to apply axial deformation. The incremental deformation level of 0.25, 0.5, 0.75, 1, 2, 4, and 6 of yielding deformation was applied to tested specimen until the failure occurred, as shown in Fig. 3. The yielding deformation is abbreviated by e_y and expressed as follow:

$$e_y = \left(\frac{F_y L}{E} \right) \quad (1)$$

where F_y = 285 MPa and 390 MPa, are the yielding stress obtained from coupon test of 1.8 mm and 3 mm thickness, respectively. The symbol L is the length of the specimen. The modulus of elasticity of steel is set to be 200 GPa.

2.2 Calculation of axially loaded capacity

The axial load capacity of CFST can be estimated from two design codes. American concrete institute (ACI) [10] proposed to superpose the capacities of the concrete and steel to predict the capacity of CFST. While AISC code [11], calculation of load capacity had been made to be identical to that of a bare steel column. The effect of slenderness ratio and proportion between steel and concrete area, however, are taken into account.

Fig.1 Test specimen with bearing plate machine.

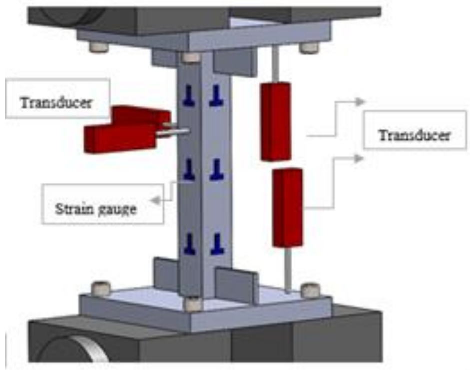


Fig.2 Installation of specimen to testing machine

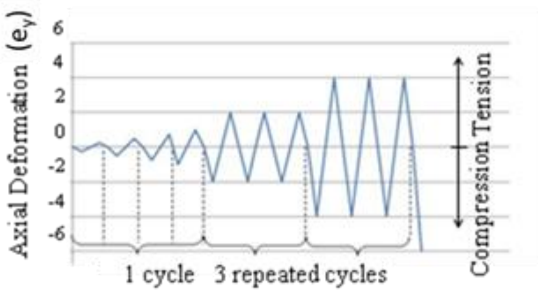


Fig.3 Cyclic loading scheme

According to ACI, the nominal capacity of CFST can be computed as follow:

$$P_n = A_s F_y + 0.85 A_c f_c' \quad (2)$$

where A_s and A_c are a respectively cross-sectional area of steel and concrete, F_y and f_c' are yield strength of steel and concrete compressive strength, respectively.

The axial load capacity in AISC code is calculated from steel tube area and compressive strength of filled composite member as given below:

$$P_{no} = P_p \quad (3)$$

In which the nominal bearing strength (P_p) depends on width to thickness ratio for the compact section as given below:

$$P_p = F_y A_s + C_2 f_c' A_c \quad (4)$$

where C_2 = 0.85 for rectangular section. Then, the design compressive strength will be determined for limit state base on slenderness as follow:

$$P_n = \begin{cases} P_{no}\left[0.658^{\frac{P_{no}}{P_e}}\right], \dfrac{P_{no}}{P_e} \le 2.25 \\ 0.877 P_e, \dfrac{P_{no}}{P_e} > 2.25 \end{cases} \quad (5)$$

where P_e is an elastic critical buckling load, calculated by following formula:

$$P_e = \pi^2 \left(EI_{eff}\right) / \left(kL\right)^2 \quad (6)$$

The effective stiffness of composite section is defined based on the following relation:

$$EI_{eff} = E_s I_s + C_3 E_c I_c, \quad C_3 = 0.6 + 2\left(\frac{A_c}{A_c + A_s}\right) \le 0.9 \quad (7)$$

Note that to compute the elastic critical buckling load according to Eq. (6), the value of k = 1 for all specimens.

For prevention of local buckling, both ACI and AISC codes recommended the minimum thickness for the square section as follow:

$$t_{min} = 0.58 b \sqrt{F_y / E} \ \text{(ACI)}, \quad t_{min} = 0.44 b \sqrt{F_y / E} \ \text{(AISC)} \quad (8)$$

where parameter b is the width of square section.

2.3 Finite element modeling

2.3.1 General

DYNA3D software package [12] was employed throughout the finite element (FE) analysis in this work. The steel tube, stiffener, and plate bearing were simulated by 8-node shell elements with 6 degree-of-freedom per node. The concrete core was modeled using 8-node brick elements with three translation degree of freedom at each node. The size of shell element and brick elements approximately equal to 10 mm. Surface-based interaction with a friction coefficient of 0.40 was used to simulate contact interface between steel tube and in-filled concrete.

Loading was applied in a displacement control mode at the top of a CFST column to simulate the axial loading condition as shown in Fig. 3. The ends of the CFST column were fixed against all degree of freedom except for the vertical displacement at the top end. The finite element model is shown in Fig. 4.

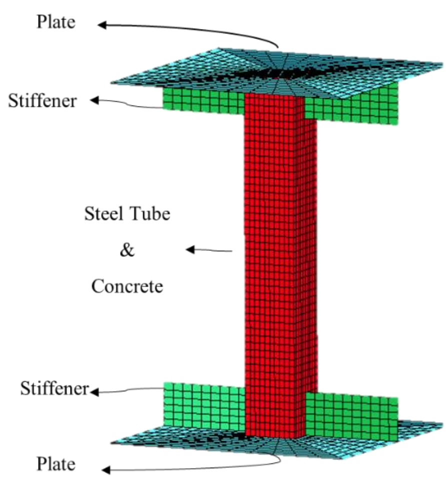
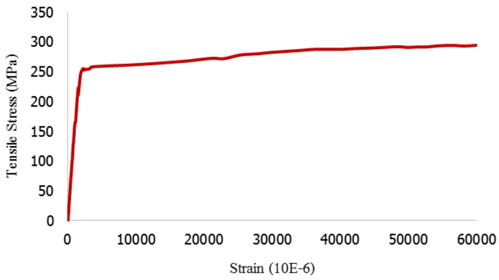


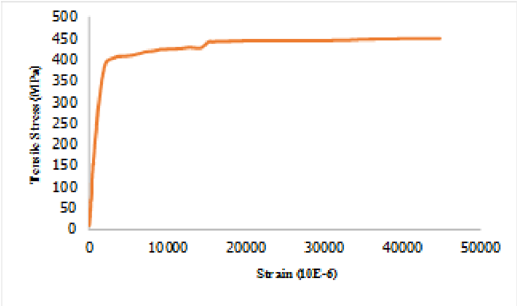
Fig.4 Finite element model

2.3.2 Material properties

To describe the stress-strain behavior of steel tube, the piece-wise linear plasticity material was used. The stress-strain curve for 1.8 mm and 3.0 mm thickness was input into the DYNA3D package as shown in Fig. 5. The Winfrith concrete model [17] was used to represent confined concrete behavior (Fig. 6).

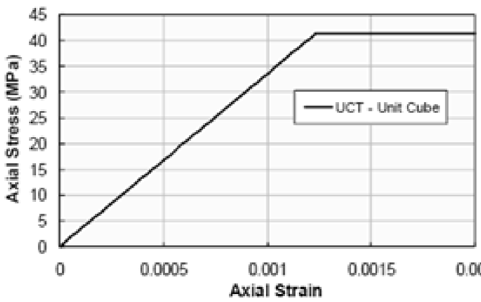


(a) Steel tube with 1.8 mm thick

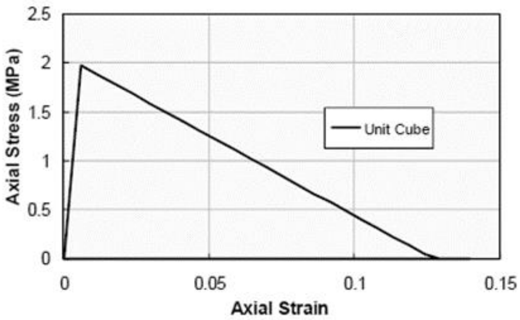


(b) Steel tube with 3 mm thick

Fig.5 Stress-strain curves for steel tube



(a) Winfrith concrete model (compression)

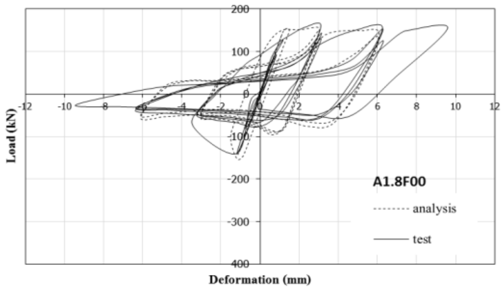


(b) Winfrith concrete model (tension)

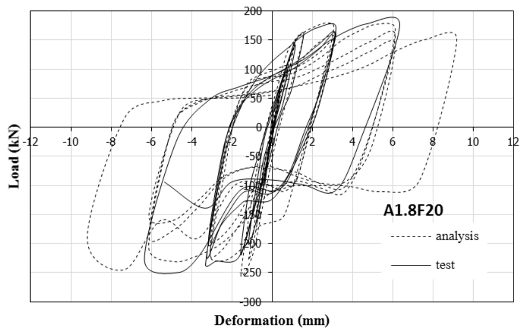
Fig.6 Stress-strain curves for concrete model [17]

3. RESULTS AND DISCUSSIONS

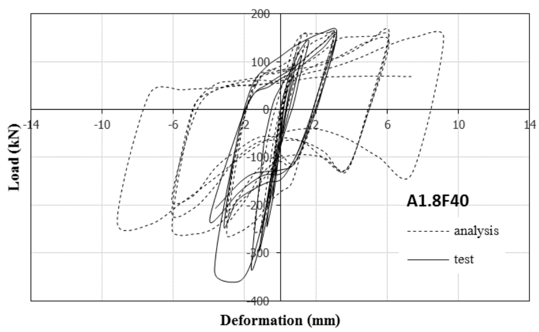
A comparison between the test results and numerical results were carried out to verify the finite element model. The behavior of the tested columns under cyclic loading can be quantitatively obtained through the consideration of load-displacement response in Figs. 7, 8 and 9. The cyclic displacement control was applied according to Fig.3. Figures. 7, 8 and 9 also show the effect of in-filled concrete on the load-deformation behavior in the form of hysteresis curves. It is clearly seen that the compressive capacity and stiffness degradation of the test columns can be improved with the in-filled concrete. Figure 7(b) shows the closer predicted value of compressive strength by finite element analysis than the value obtained from Fig. 7(c). Table 2 shows the predicted ultimate axial load obtained from experiment versus ultimate axial load predicted from FE model and design codes.



(a) Specimen A1.8F00

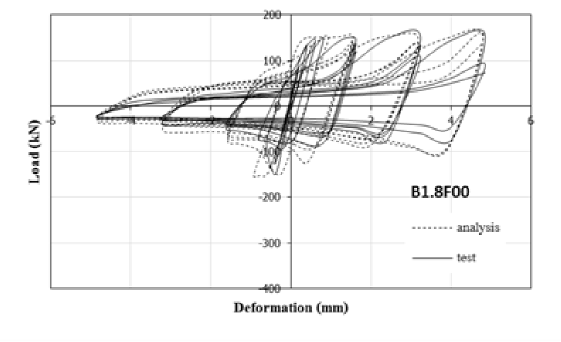


(b) Specimen A1.8F20

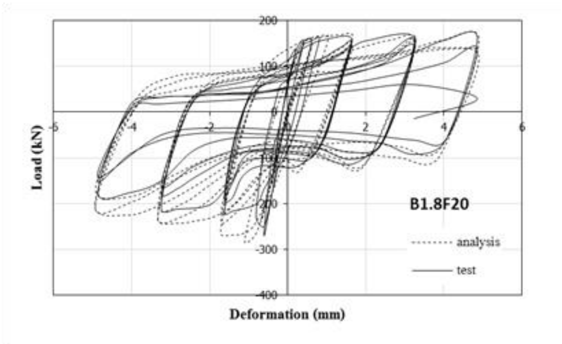


(c) Specimen A1.8F40

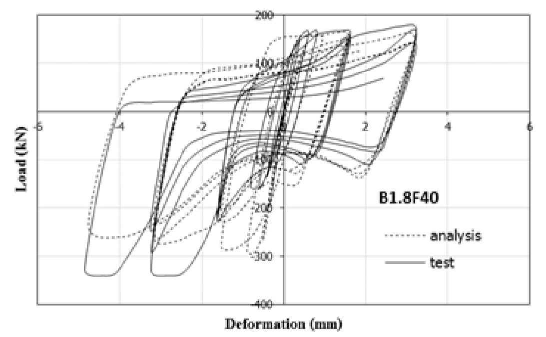
Fig.7 Load-Deflection curves and failure modes of tested specimens, L = 100 cm.



(a) Specimen B1.8F00



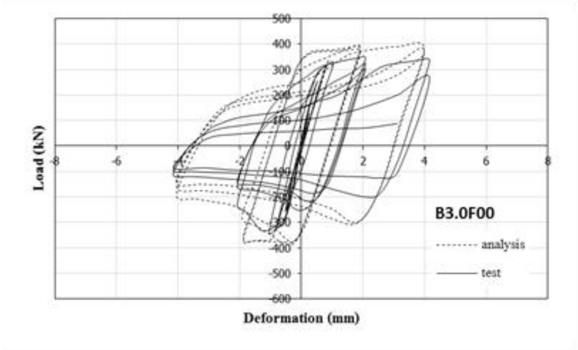
(b) Specimen B1.8F20



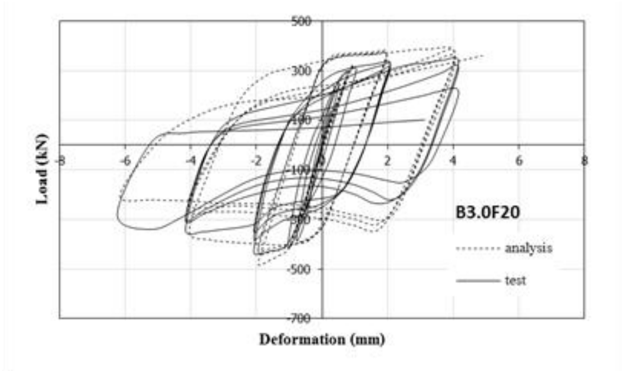
(c) Specimen B1.8F40

Fig.8 Load-Deflection curves and failure modes of tested specimens, L = 50 cm and t = 1.8 mm.

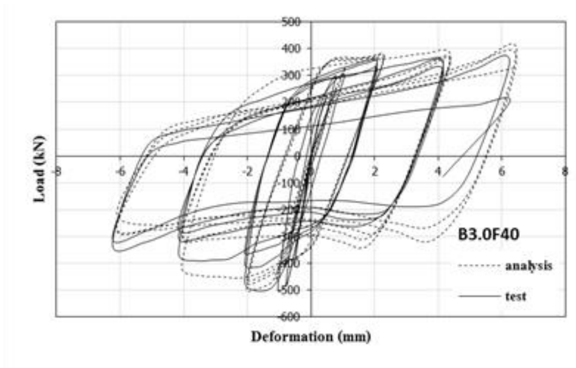
The ratio between calculated compressive strength versus test results from Table 2 is summarized in Table 3. From the above comparison, as seen in Table 3, the fairly good agreement is obtained between FE analysis and test results. Among the design codes (ACI and AISC), for the empty column, the calculation by ACI tend to give the higher strength estimation, while AISC gives closer predicted values. This is due to the inclusion of buckling effect in the AISC design codes. However, when the concrete was filled-in, the strength prediction by ACI code give closer predicted values in the case of t = 1.8 mm.



(a) Specimen B3.0F00



(b) Specimen B3.0F20



(c) Specimen B3.0F40

Fig.9 Load-Deflection curves and failure modes of tested specimens, L = 50 cm and t = 3.0 mm.

Table 2 Ultimate compressive load of CFST

Specimen	P_{test} (kN)	P_{cal} (kN)		
		AISC (2010)	ACI (2014)	FEM
A1.8F00	140	145	157	156
A1.8F20	250	222	254	255
A1.8F40	370	275	325	280
B1.8F00	145	154	157	156
B1.8F20	280	245	254	270
B1.8F40	350	312	325	295
B3.0F00	340	353	363	357
B3.0F20	440	441	456	463
B3.0F40	501	505	525	503

Moreover, the strength prediction values according to AISC and ACI codes were approximately the same with FEM when t = 3 mm.

Table 3 The compressive strength ratio of CFST

Specimen	P_{cal} / P_{test}		
	AISC (2010)	ACI (2014)	FEM
A1.8F00	1.04	1.12	1.11
A1.8F20	0.89	1.02	1.02
A1.8F40	0.74	0.88	0.76
B1.8F00	1.06	1.08	1.08
B1.8F20	0.88	0.91	0.96
B1.8F40	0.89	0.93	0.84
B3.0F00	1.04	1.07	1.05
B3.0F20	1.00	1.04	1.05
B3.0F40	1.01	1.05	1.00
average	0.95	1.01	0.99

4. CONCLUSION

Experimental and numerical studies on the response of rectangular hollow steel columns under cyclic loading were performed with nine specimens having a square tube of 75×75 mm with 1.8 mm and 3.0 mm thick cross-sections. Columns were 500 and 1000 mm in length. The main parameter is an in-filled concrete compressive strength, i.e. 0 MPa (empty), 20 MPa, and 40 MPa. The following conclusions can be drawn based on results of this study:

1) The tested results indicate that the in-filled concrete columns show better load-deformation performance; compression strength of concrete in-filled columns are higher than empty steel columns, and strength degradation was reduced when concrete is in-filled.
2) Close agreement was achieved between the test and FE results in terms of load-deformation response and ultimate strength. The average value of predicted strength from finite element analysis is slightly lower than test results.
3) The ultimate compressive strength of CFST column can be predicted with fairly accurate from AISC and ACI codes. The more conservative results were obtained if the strength of in-filled concrete increased.
4) The strength of empty columns is accurately predicted via AISC code.

5. ACKNOWLEDGEMENTS

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