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Ultimate strength of rectangular concrete-filled steel tubular (CFT) stub columns under axial compression

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Abstract. A method is proposed to estimate the ultimate strength of rectangular concrete-filled steel tubular (CFT) stub columns under axial compression. The ultimate strength of concrete core is determined by using the conception of the effective lateral confining pressure and a failure criterion of concrete under true triaxial compression, which takes into account the difference between the lateral confining pressure provided by the broad faces of the steel tube and that provided by the narrow faces of the steel tube. The longitudinal steel strength of broad faces and that of the narrow faces of the steel tube are calculated respectively due to that buckling tends to occur earlier and more extensively on the broader faces. Finally, the proposed method is verified with experimental results. Corresponding values of ultimate strength calculated by ACI (2005), AISC (1999) and GJB4142-2000 are given respectively for comparison. It is found from comparison that the proposed method shows a good agreement with the experimental results.

Keywords : concrete-filled steel tubular; ultimate strength; axial compression; failure criterion; true triaxial compression.

1. Introduction

Concrete-filled steel tubular (CFT) columns have become popular as structure members for construction of building structures in recent years. CFT columns have excellent seismic behavior such as high strength, high ductility and large energy absorption capacity.

According to a large number of experimental studies of axial behavior of CFT columns, although confinement effect could be expected in circular CFT columns, square or rectangular CFT columns show only a small increase in axial strength due to triaxial effects, even for those with large wall thickness. In addition, the studies indicate that the ductility of square or rectangular CFT columns is obviously inferior to that of circular CFT columns. Furlong (1967, 1968) conducted monotonic load test on both circular and square CFT columns. The square specimens had width-to-thickness ratios (B/t) ranging from 26 to 48, length-to-width ratios (L/B) ranging from 7 to 9. Results showed no increase in concrete strength due to confinement by steel tube. Knowles and Park (1969) investigated 12 circular and 7 square specimens with D/t of 15, 22 and 59, and L/D ratios ranging from 2 to 21. It was concluded that the confinement by steel tube only increased the ultimate strength of circular specimens. Tomii *et al.* (1977) investigated about 270 circular, octagonal and square CFT columns. Concrete confinement was

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observed in circular and many octagonal specimens at high axial loads, but square tubes provided very little confinement of the concrete because the wall of the square tube resisted the concrete pressure by plate bending, instead of the membrane-type hoop stresses. Schneider (1998) tested 3 circular, 5 square and 6 rectangular specimens with D/t ratios ranging from 17 to 50, and L/D ratios ranging from 4 to 5. A nonlinear finite element method with ABAQUS was applied to analytical study. Results indicated that circular steel tubes offered much more post-yield axial ductility than square or rectangular tube sections. All circular tubes were classified as strain hardening, while only the small D/t ratios, approximately $D/t < 20$, exhibited strain-hardening characteristics for square or rectangular tubes. Significant confinement was not present for most specimens until the axial load reaches almost 92% of the yield strength of the column. Furthermore, the square and rectangular tube walls, in most cases, did not offer significant concrete core confinement beyond the yield load of the composite column. Local wall buckling for square and rectangular tubes occurred earlier than that for circular tubes. Uy (2001) reported experiments on 19 short concrete filled high strength steel box columns with concrete cylinder strength ranging from 28 to 32 MPa. Results suggested that the EC4 model is unconservative in the prediction of axial strength. Han (2002) studied 24 rectangular CFT stub columns subjected to axial load. Results showed the strength increase of the concrete resulting from confinement effect was influenced by the cross-sectional aspect ratio and the confinement factor. By comparison, he reported that the ultimate strength of the rectangular CFT stub columns could be conservatively predicted by using the AISC (1999), AIJ (1997), EC4 (1994), and GJB4142-2000. Sakino *et al.* (2004) investigated 36 circular and 48 square CFT stub columns with concrete cylinder strength from 25 to 91 MPa, and steel tube yield strength from 262 to 835 MPa and proposed the formulae for estimating the ultimate strength of CFT columns. Test results showed an increase in axial load capacity for square specimens. Liu *et al.* (2005) tested 26 rectangular CFT columns with cross-sectional aspect ratios from 1 to 2, concrete cylinder strength ranging from 55 to 106 MPa, and steel yield strength of 300 and 495 MPa. Results indicated there was significant ductility enhancement of high-strength concrete due to the confinement by steel tube. For specimens with cross-sectional aspect ratios $D/B > 1.0$, buckling was more extensive and occurred earlier on the broader faces. Kang *et al.* (2005) studied the behavior of circular and square stub columns filled with high strength concrete and polymer cement concrete under concentric compressive load. Twenty-four specimens were tested and investigated the effects of variations in the tube shape, wall thickness and concrete type on the axial strength of the stub columns. Lue *et al.* (2007) tested 20 rectangular CFT slender columns and two rectangular hollow structural section columns. Results indicated the calculated ultimate strength of the rectangular CFT columns by EC4 (2004), AISC (1999) and ACI (2005) was much lower than that from the experiment.

Most experimental results of ultimate strength of rectangular CFT specimens available in the literature above are higher than the values determined by summing up the strengths of steel and concrete. The concrete confinement effect of rectangular CFT specimens is not constant and obviously inferior to that of circular CFT columns. As a result, confinement effect is negligible for calculating the ultimate strength of rectangular CFT columns in most design codes. Some methods considering confinement effect were proposed for calculating the ultimate strength of rectangular CFT columns (Han 2003 Liu *et al.* 2005). However, the difference in lateral confining pressures on the concrete core provided by broad faces and narrow faces is not considered. Furthermore, the longitudinal steel strength of broad faces is also different from that of narrow faces due to the difference in local buckling between broad faces and narrow faces i.e., buckling tends to occur earlier and more extensively on the broader faces (Liu *et al.* 2005). This difference should be considered reasonably. However, the methods available in the literature do not take into account the difference.

2. Brief review of methods for calculation of ultimate strength of rectangular CFT stub columns

2.1 Method proposed by EC4 (2004)

EC4 (2004) predicts the axial load capacity (N_c) of CFT stub columns by Eq. (1).

$$N_c = A_s F_y / \gamma_s + 0.85 A_c \ f_c' / \gamma_c, \qquad \gamma_s = 1.1, \quad \gamma_c = 1.5 \tag{1}$$

where γ_s is the partial safety factor for structural steel and γ_c is the partial safety factor for concrete.

2.2 Method proposed by ACI (2005)

ACI (2005) estimates the axial load capacity of CFT columns in a similar method as for reinforced concrete columns.

$$N_c = A_s F_y + 0.85 A_c f_c' \tag{2}$$

2.3 Method proposed by AISC (1999)

The AISC (1999) method for the design of CFT columns is essentially identical to that for steel columns, shown in Eq. (3).

$$N_c = A_s F_{cr} \tag{3}$$

where $F_{cr} = (0.658^{\lambda_c^2}) F_{my}$ for $\lambda_c \le 1.5$; $F_{cr} = (0.877 / \lambda_c^2) F_{my}$ for $\lambda_c > 1.5$; $\lambda_c = (L / r_s \pi) \sqrt{F_{my} / (E_s + 0.4 E_c A_c / A_s)}$; $F_{my} = f_y + 0.85 f_c' A_c / A_s$; r_s is radius of gyration of steel tube; L is length of column; E_s is modulus of elasticity of steel and E_c is modulus of elasticity of concrete.

2.4 Method proposed by GJB4142-2000

Methods proposed by EC4 (2004), ACI (2005) and AISC (1999) do not consider the confinement effect of concrete for rectangular CFT columns. However, GJB4142-2000 (2001) takes into account the confinement effect of concrete with the so-called constraining factor ξ and regards the concrete core and the steel tube as an entity. The ultimate strength can be expressed as

$$N_c = A_{sc} f_{scy} \tag{4}$$

$$A_{sc} = A_s + A_c \tag{5}$$

$$f_{scy} = (1.212 + B\xi + C\xi^2) f_{ck} \tag{6}$$

where $B = 0.1381 f_y / 235 + 0.7646$; $C = -0.0727 f_{ck} / 20 + 0.0216$; $\xi = A_s f_y / A_c f_{ck}$; f_{ck} is characteristic strength of concrete defined as $f_{ck} = 0.67 f_{cu}$.

2.5 Method proposed by Uy (2000)

Uy (2000) suggested a method for the ultimate strength of concrete filled steel box columns under pure compression, which considers local buckling of steel tube. The ultimate strength can be expressed as

$$N_c = f_c A_c + f_y A_{se} \tag{7}$$

where A_{se} represents the effective steel area. This effective steel area depends on an effective width model illustrated in Fig.1 and given by

$$\frac{b_e}{b} = \alpha \sqrt{\frac{\sigma_{ol}}{\sigma_y}} \tag{8}$$

where $\alpha = 0.65$ according the Australian Standard AS4100; σ_{ol} and σ_y are respectively elastic local buckling stress and yield stress of steel.

2.6 Assessment of the methods above

EC4 (2004), ACI (2005), AISC (1999) and the method proposed by Uy do not take into account concrete confinement effect, resulting in conservative predictions for the ultimate strength of rectangular CFT columns under axial compression. GJB4142-2000 considers the concrete confinement effect. However, GJB4142-2000 does not consider the difference in confinement effects on the concrete core provided by broad faces and narrow faces. Furthermore, the methods above do not take into account the difference in the longitudinal steel strength between broad faces and narrow faces due to the different extent of buckling between broad faces and narrow faces.

3. Proposed method

A method is proposed for calculating the ultimate strength of rectangular CFT stub columns under

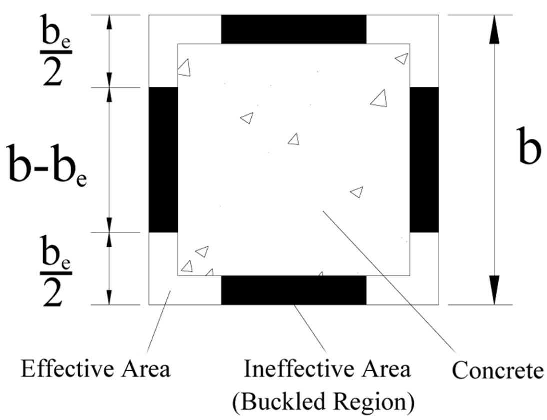


Fig. 1 Effective width of concrete filled steel box columns

axial compression, which considers both the difference in the longitudinal steel strength between broad faces and narrow faces and the difference in lateral confining pressure on the concrete core. The formula of the ultimate strength of rectangular CFT stub columns is expressed as

$$N_u = A_{s1} f_{a1} + A_{s2} f_{a2} + A_c f'_{cc} \tag{9}$$

where f_{a1} and f_{a2} are respectively the longitudinal strength of steel of the broad faces and the longitudinal strength of steel of the narrow faces of steel tube; A_{s1} and A_{s2} are respectively area of the broad faces and area of the narrow faces; f'_{cc} is the longitudinal strength of concrete.

The concrete core is under true triaxial compression at the ultimate strength. To determine the confined concrete compressive strength f'_{cc}, a failure criterion for concrete under true triaxial compression adopted by Chinese Standard GB50010-2002 (2002) is used and given by the equations as follows.

$$\tau_0 = 6.9638\left(\frac{0.09-\sigma_0}{c-\sigma_0}\right)^{0.9297} \tag{10}$$

$$c = 12.2445(\cos 1.5\alpha)^{1.5} + 7.3319(\sin 1.5\alpha)^2 \tag{11}$$

$$\sigma_0 = \sigma_{oct} / f'_{co}\,;\tau_0 = \tau_{oct} / f'_{co} \tag{12}$$

$$\sigma_{oct} = \frac{f_{l1} + f_{l2} + f'_{cc}}{3} \tag{13}$$

$$\tau_{oct} = \frac{\sqrt{(f_{l1} - f_{l2})^2 + (f_{l2} - f'_{cc})^2 + (f'_{cc} - f_{l1})^2}}{3} \tag{14}$$

$$\cos\varphi = \frac{2f_{l1} - f_{l2} - f'_{cc}}{3\sqrt{2}\,\tau_{oct}} \tag{15}$$

where σ_{oct} and τ_{oct} are the normal and shear octahedral stresses, respectively; f'_{co} is compressive strength of unconfined concrete; The rotational variable φ defines the direction of the deviatoric component on the octahedral plane; f_{l1} and f_{l2} are respectively effective lateral confining pressure along the broad faces and effective lateral confining pressure along the narrow faces.

The concrete core confined by rectangular steel tube is similar to concrete confined by rectangular hoops i.e., they can be characterized as confined concrete in essence. The difference in the behavior between concrete confined by rectangular steel tube and concrete confined by rectangular hoops lies in the difference in concrete confinement effects. As a result, an approach similar to that used by Mander *et al.* (1988) is used to determine the effective lateral confining pressure. f_{l1} and f_{l2} are respectively given by

$$f_{l1} = k_e \cdot f'_{l1} \tag{16}$$

$$f_{l2}=k_e \cdot f'_{l2} \tag{17}$$

in which f'_{l1} and f'_{l2} are respectively the lateral confining stress along the broad faces and the lateral confining pressure along the narrow faces. Assumed to be uniformly distributed over the surface of the concrete core, f'_{l1} and f'_{l2} can result from Fig. 2 according to the equilibrium of forces as follows

$$f'_{l1}\,(B-2t)-2f_{sr1}\cdot t=0 \qquad \text{(along the broad faces)} \tag{18}$$

$$f'_{l2}\,(B-2t)-2f_{sr2}\cdot t=0 \qquad \text{(along the narrow faces)} \tag{19}$$

From Eq. (18), the lateral confining stress along the broad faces f'_{l1} is given by Eq. (20).

$$f'_{l1}=\frac{2f_{sr1}}{B/t-2} \tag{20}$$

Similarly, the lateral confining stress along the narrow faces f'_{l2} is given by Eq. (21).

$$f'_{l2}=\frac{2f_{sr2}}{D/t-2} \tag{21}$$

The confinement effective coefficient k_e is divided into the lever confinement effective coefficient k_{e1} and the longitudinal confinement effective coefficient k_{e2} herein.

$$k_e=k_{e1}k_{e2} \tag{22}$$

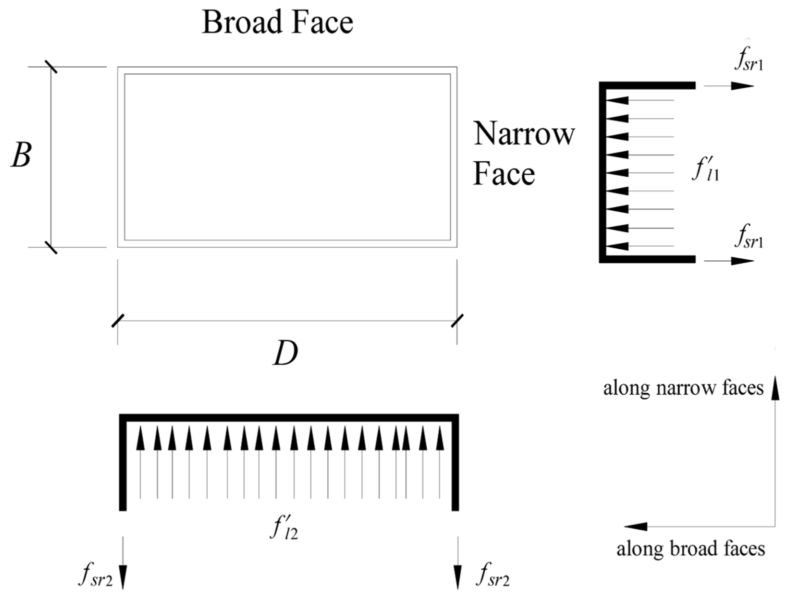


Fig. 2 Lateral confinement of concrete core

where $k_{e2} = 1$ for rectangular CFT columns due to that the confinement effect of steel tube is longitudinally continuous and constant while Mander *et al*. (1988) suggested $k_{e2} < 1$ for rectangular reinforced concrete columns due to that the stirrups or hoops are arranged at intervals along the axis of the column; k_{e1} takes the form Mander *et al*. (1988) as

$$k_{e1}=\frac{A_{e1}}{A_{cc1}} \tag{23}$$

where A_{e1} is the level area of an effectively confined concrete core; $A_{cc1}$is the level area of a concrete core. The arching action shown in Fig. 3 is assumed to occur in the form of a second-degree parabola with an initial tangent angle θ. So that A_{e1} and A_{cc1} are given by

$$A_{e1}=(B-2t)(D-2t)-2A_I-2A_{II} \tag{24}$$

$$A_{cc1}=(B-2t)(D-2t) \tag{25}$$

where A_I and A_{II} are the area of the parabolas indicated in Fig. 3.

$$A_I=\frac{(B-2t)^2\tan\theta_1}{6} \tag{26}$$

$$A_{II}=\frac{(D-2t)^2\tan\theta_2}{6} \tag{27}$$

From Eqs. (23) ~ (27), k_{e1} is given by

$$k_{e1}=1-\frac{(B-2t)\tan\theta_1}{3(D-2t)}-\frac{(D-2t)\tan\theta_2}{3(B-2t)} \tag{28}$$

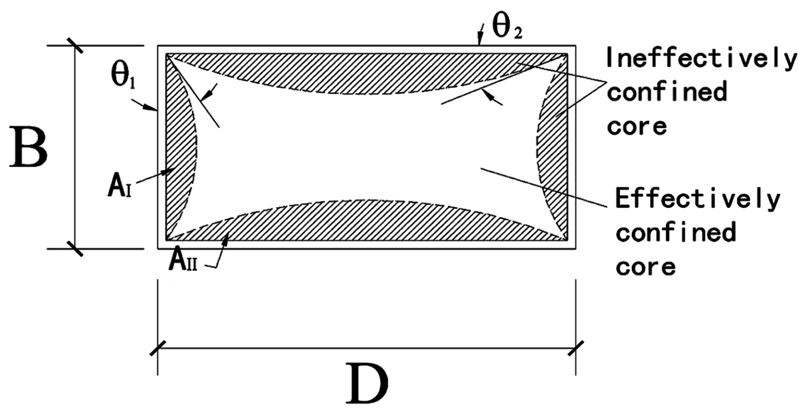


Fig. 3 Effectively confined concrete core for rectangular CFT columns

For square section i.e., applying $B = D$ and $\theta_1 = \theta_2$ to Eq. (28) results in

$$k_{e1} = 1 - \frac{2\tan\theta_1}{3} \tag{29}$$

Then, the confinement effective coefficient k_e for rectangular CFT columns is given by

$$k_e = 1 - \frac{(B-2t)\tan\theta_1}{3(D-2t)} - \frac{(D-2t)\tan\theta_2}{3(B-2t)} \tag{30}$$

Mander *et al.* (1988) suggested the initial tangent angle $\theta_1 = \theta_2 = 45^\circ$for rectangular reinforced concrete columns. However, it was found the initial tangent angle θ_1 and θ_2 for the rectangular CFT stub columns cannot be taken as a constant and they are mainly sensitive to the initial tangent angle coefficients ζ_1 and ζ_2, respectively. Based on a double nonlinear regression of experimental data from Han (2002), the empirical equations for θ_1 and θ_2 are given by

$$\theta_1 = -0.078\zeta_1^2 + 4.8\zeta_1 - 22.6 \tag{31}$$

$$\theta_2 = -0.078\zeta_2^2 + 4.8\zeta_2 - 22.6 \tag{32}$$

$$\zeta_1 = \frac{f_y}{B/t} \tag{33}$$

$$\zeta_2 = \frac{f_y}{D/t} \tag{34}$$

where ζ_1 and ζ_2 are the initial tangent angle coefficients.

It is assumed that the steel stress at ultimate state obey the Von Mises criteria given by

$$f_{a1}^2 - f_{a1} f_{sr1} + f_{sr1}^2 = f_y^2 \tag{35}$$

For the local buckling strength of a steel plate f_b in CFT columns, a relationship proposed by Ge and Usami (1994) is applied herein.

$$\frac{f_b}{f_y} = \frac{1.2}{R_1} - \frac{0.3}{R_1^2} \leq 1.0 \tag{36}$$

which is ensured $R_1 \geq 0.85$; where R_1 is the depth-to- thickness ratio parameter, defined by

$$R_1 = \frac{D}{t}\sqrt{\frac{12(1-\nu^2)}{4\pi^2}}\sqrt{\frac{f_y}{E_a}} \tag{37}$$

Then from Eq. (36), the stress states of steel tubes can be determined in the two cases as follows. For $R_1 \geq 0.85$, applying $f_{a1} = f_b$ to Eqs.(35) and (36) results in

$$f_{a1} = \left(\frac{1.2}{R_1} - \frac{0.3}{R_1^2}\right) f_y \tag{38}$$

$$f_{sr1} = \frac{f_{a1} - (\sqrt{4f_y^2 - 3f_{a1}^2})}{2} \tag{39}$$

If the calculated values of f_{a1} by Eq. (38) is greater than $0.89f_y$, $f_{a1} = 0.89f_y$ and $f_{sr1} = -0.19f_y$

For $R_1 < 0.85$ the effects of local buckling can be ignored (Ge *et al*. 1994). In this case, an expression deduced by Sakino *et al*. (2004) is used as follows.

$$f_{a1} = 0.89f_y \tag{40}$$

$$f_{sr1} = -0.19f_y \tag{41}$$

Similarly, f_{a2} and f_{sr2} can be determined as follows.
For $R_2 \geq 0.85$,

$$f_{a2} = \left(\frac{1.2}{R_2} - \frac{0.3}{R_2^2}\right) f_y \tag{42}$$

$$f_{sr2} = \frac{f_{a2} - \sqrt{4f_y^2 - 3f_{a2}^2}}{2} \tag{43}$$

where R_2 is the width-to-thickness ratio parameter, defined by

$$R_2 = \frac{B}{t}\sqrt{\frac{12(1-\nu^2)}{4\pi^2}}\sqrt{\frac{f_y}{E_a}} \tag{44}$$

If the calculated values of f_{a2} by Eq.(42) is greater than $0.89f_y$, $f_{a2} = 0.89f_y$ and $f_{sr2} = -0.19f_y$.

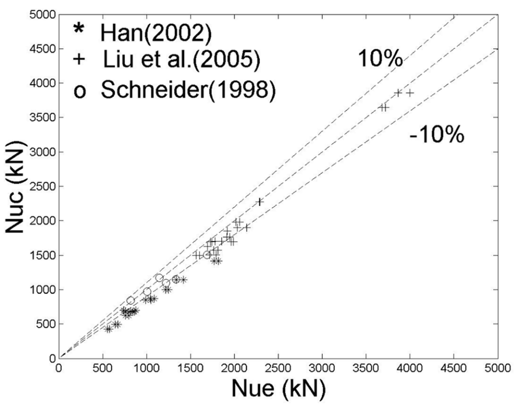
For $R_1 < 0.85$

$$f_{a2} = 0.89f_y \tag{45}$$

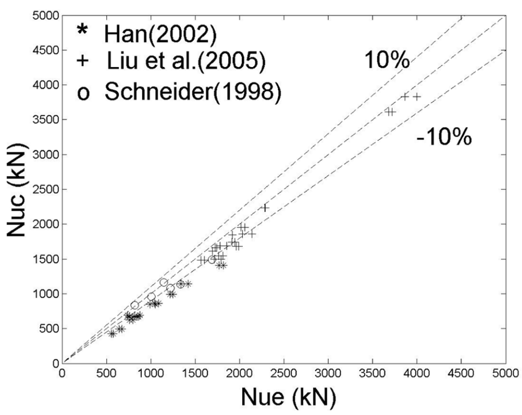
$$f_{sr2} = -0.19 f_y \tag{46}$$

4. Comparison between experiments, design codes and proposed method

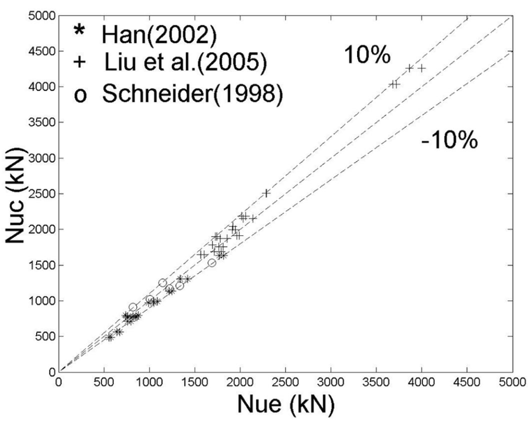
The experimental results of 56 rectangular CFT specimens available in Schneider (1998), Han (2002) and Liu *et al.* (2005) are compared with the calculated values by ACI (2005), ASIC (1999), GJB4142-2000 and proposed method, displayed in Fig. 4 (a), (b), (c), (d), respectively and detail comparison listed in Table 1. As shown in Fig 4 and Table1, ACI(2005) and ASIC(1999) conservatively predict the ultimate strength of the rectangular specimens by 11% and 12% respectively. GJB4142-2000 and the proposed method give a slightly lower capacity than the experimental results with mean value 0.989 and 0.988 respectively, standard deviation 0.08 and 0.06. The proposed method is the best predictor with lowest standard deviation due to that it can considers both the difference in the longitudinal steel strength between broad faces and narrow faces and the difference in lateral confining pressure on the concrete core.



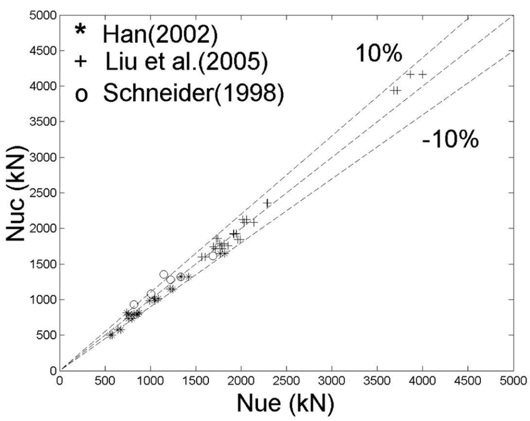
(a) Comparison between test and ACI



(b) Comparison between test and ASIC



(c) Comparison between test and GJB



(d) Comparison between test and proposed method

Fig. 4 Comparison between tests, design codes and proposed method

Table 1 Comparison of test results of rectangular specimens with EC4, ASIC, GBJ and proposed model

No	$D \times B \times t$ (mm)	L (mm)	f_{ck} (MPa)	f'_c (MPa)	f_y (MPa)	N_{ue} (kN)	ACI(2005)		AISC(1999)		GJB4142-2000		Proposed model		Test data resources
							N_{ue}(kN)	N_{uc}/ N_{ue}	N_{uc}(kN)	N_{uc}/ N_{ue}	N_{uc}(kN)	N_{uc}/N_{ue}	N_{uc}(kN)	N_{uc}/N_{ue}	
A1	120×120×5.8	360	62.31	83	300	1697	1624	0.957	1615	0.952	1787	1.053	1742	1.027	
A2	120×120×5.8	360	77.05	106	300	1919	1854	0.966	1842	0.960	2040	1.063	1920	1.001	
A3-1	200×200×5.8	600	62.31	83	300	3996	3856	0.965	3832	0.959	4263	1.067	4166	1.043	
A3-2	200×200×5.8	600	62.31	83	300	3862	3856	0.998	3832	0.992	4263	1.104	4166	1.079	
A4-1	130×100×5.8	390	62.31	83	300	1601	1498	0.936	1485	0.928	1645	1.028	1597	0.998	
A4-2	130×100×5.8	390	62.31	83	300	1566	1498	0.957	1485	0.948	1645	1.050	1597	1.020	
A5-1	130×100×5.8	390	77.05	106	300	1854	1703	0.919	1686	0.909	1872	1.010	1757	0.948	
A5-2	130×100×5.8	390	77.05	106	300	1779	1703	0.957	1686	0.948	1872	1.052	1757	0.988	
A6-1	220×170×5.8	660	62.31	83	300	3684	3646	0.989	3610	0.980	4032	1.095	3936	1.068	
A6-2	220×170×5.8	660	62.31	83	300	3717	3646	0.981	3610	0.971	4032	1.085	3936	1.059	
A7-1	180×100×5.8	540	62.31	83	300	2059	1984	0.964	1952	0.948	2189	1.063	2126	1.033	
A7-2	180×100×5.8	540	62.31	83	300	2019	1984	0.983	1952	0.967	2189	1.084	2126	1.053	
A8-1	180×100×5.8	540	77.05	106	300	2287	2275	0.995	2233	0.976	2506	1.096	2354	1.029	Liu *et al.* (2005)
A8-2	180×100×5.8	540	77.05	106	300	2291	2275	0.993	2233	0.975	2506	1.094	2354	1.028	
A9-1	120×120×4	360	46.9	55	495	1793	1505	0.839	1496	0.834	1687	0.941	1715	0.957	
A9-2	120×120×4	360	46.9	55	495	1718	1505	0.876	1496	0.871	1687	0.982	1715	0.998	
A10-1	150×100×4	450	46.9	55	495	1815	1569	0.865	1548	0.853	1758	0.969	1785	0.984	
A10-2	150×100×4	450	46.9	55	495	1763	1569	0.890	1548	0.878	1758	0.997	1785	1.013	
A11-1	180×90×4	540	46.9	55	495	1725	1697	0.984	1659	0.962	1900	1.101	1858	1.077	
A11-2	180×90×4	540	46.9	55	495	1742	1697	0.974	1659	0.952	1900	1.091	1858	1.067	
A12-1	130×130×4	390	46.9	55	495	1963	1694	0.863	1683	0.857	1915	0.976	1842	0.938	
A12-2	130×130×4	390	46.9	55	495	1988	1694	0.852	1683	0.847	1915	0.963	1842	0.927	
A13-1	160×110×4	480	46.9	55	495	1947	1762	0.905	1741	0.894	1993	1.024	1927	0.990	
A13-2	160×110×4	480	46.9	55	495	1912	1762	0.922	1741	0.911	1993	1.042	1927	1.008	
A14-1	190×100×4	570	46.9	55	495	2035	1900	0.934	1861	0.915	2149	1.056	2081	1.023	
A14-2	190×100×4	570	46.9	55	495	2138	1900	0.889	1861	0.870	2149	1.005	2081	0.973	
rc1-1	100×100×2.86	300	39.73	49.22	227.7	760	625	0.822	623	0.820	712	0.937	733	0.965	
rc1-2	100×100×2.86	300	39.73	49.22	227.7	800	625	0.781	623	0.779	712	0.890	733	0.916	Han (2002)
rc2-1	120×120×2.86	360	39.73	49.22	227.7	992	852	0.859	849	0.856	971	0.979	989	0.997	
rc2-2	120×120×2.86	360	39.73	49.22	227.7	1050	852	0.811	849	0.809	971	0.925	989	0.942	

Table 1 Comparison of test results of rectangular specimens with EC4, ASIC, GBJ and proposed model(Continue...)

No	$D \times B \times t$ (mm)	L (mm)	f_{ck} (MPa)	f'_c (MPa)	f_y (MPa)	N_{ue} (kN)	ACI(2005)		AISC(1999)		GJB4142-2000		Proposed model		Test data resources
							N_{ue}(kN)	N_{uc}/N_{ue}	N_{ue}(kN)	N_{uc}/N_{ue}	N_{ue}(kN)	N_{uc}/N_{ue}	N_{ue}(kN)	N_{uc}/N_{ue}	
rc3-1	110×110×2.86	330	39.73	49.22	227.7	844	677	0.802	675	0.800	772	0.915	792	0.938	Han (2002)
rc3-2	110×110×2.86	330	39.73	49.22	227.7	860	677	0.787	675	0.785	772	0.898	792	0.921	
rc4-1	150×135×2.86	450	39.73	49.22	227.7	1420	1144	0.806	1140	0.803	1306	0.920	1320	0.930	
rc4-2	150×135×2.86	450	39.73	49.22	227.7	1340	1144	0.854	1140	0.851	1306	0.975	1320	0.985	
rc5-1	90×70×2.86	270	39.73	49.22	227.7	554	428	0.773	426	0.769	487	0.879	502	0.906	
rc5-2	90×70×2.86	270	39.73	49.22	227.7	576	428	0.743	426	0.740	487	0.845	502	0.872	
rc6-1	100×75×2.86	300	39.73	49.22	227.7	640	494	0.772	492	0.769	562	0.878	579	0.905	
rc6-2	100×75×2.86	300	39.73	49.22	227.7	672	494	0.735	492	0.732	562	0.836	579	0.862	
rc7-1	120×90×2.86	360	39.73	49.22	227.7	800	669	0.836	667	0.834	763	0.954	780	0.975	
rc7-2	120×90×2.86	360	39.73	49.22	227.7	760	669	0.880	667	0.878	763	1.004	780	1.026	
rc8-1	140×105×2.86	420	39.73	49.22	227.7	1044	869	0.832	866	0.830	992	0.950	1012	0.969	
rc8-2	140×105×2.86	420	39.73	49.22	227.7	1086	869	0.800	866	0.797	992	0.913	1012	0.932	
rc9-1	150×115×2.86	450	39.73	49.22	227.7	1251	997	0.797	994	0.795	1138	0.910	1146	0.916	
rc9-2	150×115×2.86	450	39.73	49.22	227.7	1218	997	0.819	994	0.816	1138	0.934	1146	0.941	
rc10-1	160×120×7.6	480	39.73	49.22	194	1820	1416	0.778	1412	0.776	1635	0.898	1647	0.905	
rc10-2	160×120×7.6	480	39.73	49.22	194	1770	1416	0.800	1412	0.776	1635	0.924	1647	0.931	
rc11-1	130×85×2.86	390	39.73	49.22	227.7	760	685	0.901	682	0.897	781	1.028	797	1.049	
rc11-2	130×85×2.86	390	39.73	49.22	227.7	820	685	0.835	682	0.832	781	0.952	797	0.972	
rc12-1	140×80×2.86	420	39.73	49.22	227.7	880	696	0.791	694	0.789	794	0.902	815	0.926	
rc12-2	140×80×2.86	420	39.73	49.22	227.7	740	696	0.941	694	0.938	794	1.073	815	1.101	
R1	152.3×76.6×3	608	25.02	30.454	430	819	842	1.028	833	1.017	914	1.116	930	1.135	Schneider (1998)
R2	152.8×76.5×4.47	608	21.59	26.044	383	1006	970	0.964	960	0.954	1018	1.012	1084	1.077	
R3	152.4×101.8×4.32	608	21.59	26.044	413	1144	1173	1.025	1161	1.015	1253	1.095	1355	1.107	
R4	152.7×102.8×4.47	608	19.84	23.805	365	1224	1094	0.894	1084	0.886	1168	0.954	1280	1.046	
R5	151.4×101.3×5.72	608	19.84	23.805	324	1335	1149	0.861	1139	0.853	1213	0.909	1320	0.921	
R6	151.4×102.1×7.34	608	19.84	23.805	358	1691	1504	0.889	1491	0.882	1528	0.904	1613	0.954	
Mean								0.89		0.88		0.99		0.99	
SD								0.08		0.08		0.08		0.06	

5. Conclusions

This paper proposed a method for calculating the ultimate strength of rectangular CFT stub columns under axial compression. Difference in the longitudinal steel strength between broad faces and narrow faces and the difference in lateral confining pressure on the concrete core are considered. The experimental results of 56 rectangular CFT specimens are compared with the calculated values by ACI (2005), GJB4142-2000, ASIC (1999) and proposed method. It is found from comparison that the proposed method shows a good agreement with the test results.

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Nomenclature

A_a	Area of steel tube
A_{s1}	Area of the broad faces of steel tube
A_{s2}	Area of the narrow faces of steel tube
A_c	Area of concrete core
D	Depth of rectangular steel tube
B	Width of rectangular steel tube
t	Tube wall thickness
L	Length of steel tube
E_a	Elastic modulus of steel tube
E_c	Elastic modulus of concrete
f_{cu}	Characteristic 28-day cubic strength of concrete
f_{ck}	Characteristic strength of concrete defined as $f_{ck} = 0.67f_{cu}$
f_y	Yield strength of steel tube
N_{uc}	Calculated ultimate strength of composite columns
N_{ue}	Experimental ultimate strength of composite columns

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