MECHANICAL BEHAVIOR AND ULTIMATE STRENGTH OF CIRCULAR CFT COLUMNS SUBJECTED TO AXIAL COMPRESSION LOADS

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ABSTRACT :

In this paper, a simple formula for predicting the axial capacity of circular concrete-filled steel tube (CFT) stub columns is proposed. The concrete confinement, which depends mainly on the ratio of the external diameter of the steel tube to the plate thickness, the yield stress of the steel tube and the unconfined compressive strength of the filled concrete, is empirically deduced. A comparison with the experimental data published in the literature indicates that the present method provides more efficient representation of the ultimate strength of circular CFT stub columns than the existing codes, such as ACI-2005, AIJ-2001, Eurocode-2004 and DL/T-1999.

KEYWORDS: Composite columns, Concrete, Steel, Tubes, Capacity, Confinement

1. INTRODUCTION

Concrete-filled steel tubular (CFT) columns are being more widely used in the construction of high-rise buildings, bridges, subway platforms, and barriers. Their usage provides excellent static and earthquake-resistant properties, such as high strength, high ductility, high stiffness, and large energy-absorption capacity. CFT columns provide the benefits of both steel and concrete: a steel tube surrounding a concrete column not only assists in carrying axial load but also confines to the concrete. Furthermore, it eliminates the permanent formwork, which reduces construction time, while the concrete core takes the axial load and prevents or delays local buckling of the steel tube.

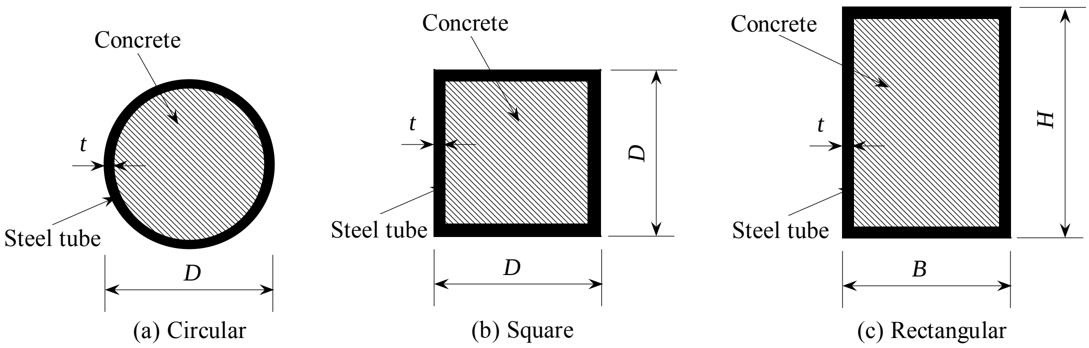

Figure 1 Concrete-filled steel tubular columns

There are many types of CFT columns, as illustrated in Figure 1. Of interest here are short circular CFT columns, which are considered to offer much more post-yield axial ductility than square and rectangular tube sections (Schneider 1998), and are the more commonly used type in many modern structures. Research on circular CFT stub columns has been ongoing worldwide for decades, and significant contributions have been made by many researchers. A series of tests on the behavior of circular thin-walled steel tubes have been carried out by O'shea and Bridge (1997, 2000), who concluded that a steel tube with a diameter-to-thickness ratio greater than 55 and filled with 110-120 MPa high-strength concrete provides insignificant confinement to the concrete. Brauns (1999) stated that effect of confinement exists at high stress levels when the structural steel acts in tension and the concrete in compression. Fourteen short CFT columns subjected to axial loads were tested by Schneider (1998) to investigate the effect of the tube shape and steel tube plate thickness on the composite column strength. Huang et al. (2002) tested 17 CFT columns specimens with a higher diameter-to-thickness ratio. Sakino et al. (2004) studied the effect of steel tube tensile strength and concrete strength on the behavior of composite columns. Giakoumelis and Lam (2004) carried out 15 tests on circular CFT columns and investigated the effects of the steel tube plate thickness, the bond between the steel tube and concrete, and the concrete confinement on the behavior of these columns.

On the other hand, success has been achieved in developing accurate models for the concrete confinement and the interaction between the steel tube and the concrete core. Schneider (1998) and Hu et al. (2003) developed a 3-D nonlinear finite element model for CFT circular columns, in which the ABAQUS program was used. The concrete confinement was achieved by matching the numerical results by trial and error using a parametric study. A multi-linear stress-strain curve for the steel tube was used and the interface between concrete and steel tube was modeled to improve the analytical results of the ABAQUS program (Ellobody et al. 2006). The accuracy of these analytical methods has been verified against the results of the tests. However, a simple relationship between the concrete confinement and the diameter-to-thickness ratio, the strength of the steel tube and filled concrete has not been established up to now. Recently, a 3-D FEM simulation for fracture behavior of CFT in axial compression was conducted by Shibata and Tachibana (2006).

In general, these studies have been shown that with the advent of high-strength steel and the production of high-strength concrete using conventional materials with careful quality control, high-strength CFT columns are both technically and economically feasible. However, they are scarcely adopted in the construction industry, mainly due to the lack of understanding of their structural behavior and reliable design recommendations (Liu 2005). The present design codes, such as Eurocode 4 (2004), AIJ (2001), ACI (2005), and DL/T (1999) have some limitations in applications concerning material strength and the diameter-to-thickness ratio of circular steel tubes. In order to expand the applications of high strength and diameter-to-thickness ratio CFT columns, it is necessary to have a simple and accurate formula to understand the concrete confinement and predict the axial capacity of circular concrete-filled steel tube stub columns with not only normal strength but also with high-strength concrete and steel.

The intention of the present paper is to clarify the effects of the diameter-to-thickness ratio, the strength of the concrete and steel tube on the concrete confinement, and to present a simple and accurate formula to predict the axial capacity of circular CFT stub columns with both normal and high-strength concrete and steel. A comparison with the recent experimental results (O'Shea and Bridge 2000; Sakino et al. 2004; Han et al. 2005; Yamamoto et al. 2002) indicates that the present formula can effectively predict the ultimate capacity of normal and high-strength circular CFT stub columns.

2. A SIMPLE FORMULA FOR AXIALLY CAPACITY OF CIRCULAR CFT STUB COLUMNS

2.1. Mechanical Behavior of Stub Columns

It is known that stub columns concentrically loaded on the entire section are significantly affected by the difference between the values of Poisson's ratio of the steel tube, v_s, and the concrete core, v_c (Gardner and Jacobson 1967). In the initial stage of loading, Poisson's ratio for the concrete is lower than that for steel; therefore, the steel tube expands faster in the radial direction than the concrete core, i.e., the steel does not restrain the concrete core. Provided the bond between the steel and concrete does not break, the initial circumferential steel hoop stresses are compressive and the concrete is under lateral tension, as shown in Figure 2a; otherwise, a separation between the steel tube and concrete core occurs. As the load increases and the

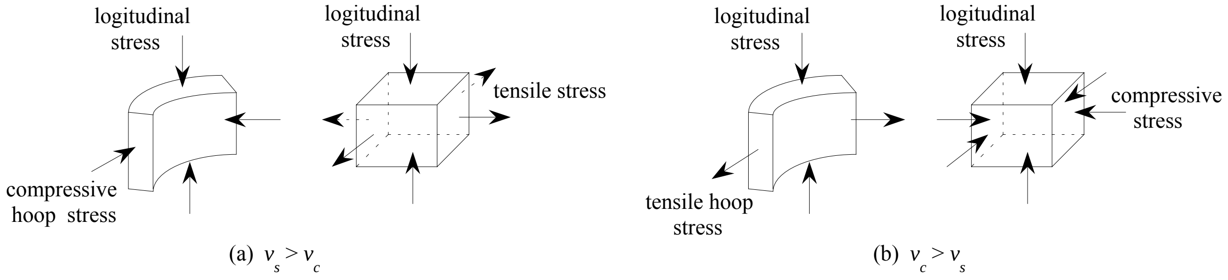

Figure 2 Stress conditions in steel tube and concrete core at different stages of loading

compressed concrete starts to plasticize, the lateral deformations of the concrete catch up with those of the steel and, with a further increase in load, the steel tube restrains the concrete core and the hoop stresses in the steel become tensile, as shown in Figure 2b. At this stage and later, the concrete core is stressed triaxially and the steel tube biaxially. This phenomenon results in an increase of axial compressive load capacity. Because of the presence of the hoop tension, the steel tube cannot sustain the plastic resistance in an axial direction. The bond strength has no effect on structural behavior because there is no relative movement between the concrete core and the steel tube.

2.2. Formulation of Axial Capacity of Circular CFT Stub Columns

The stress state for a circular CFT stub column is shown in Figure 3. When the CFT section is under the ultimate compression force N_{cu}, the concrete in a circular CFT section is subjected to axial stress σ_{cc} and lateral pressure σ_r, and the steel tube is subjected to axial stress σ_{sz} and ring tension stress $\sigma_{s\theta}$. From Figure 3, one can easily understand that the axial compressive strength N_{cu} can be given by

$$N_{cu} = A_c\sigma_{cc} + A_S\sigma_{sz} \tag{2.1}$$

where A_s = the cross-sectional area of the steel tube; A_c = the cross-sectional area of the concrete; σ_{cc} = the strength of the confined concrete; and σ_{sz} = the ultimate value of the axial stress of steel.

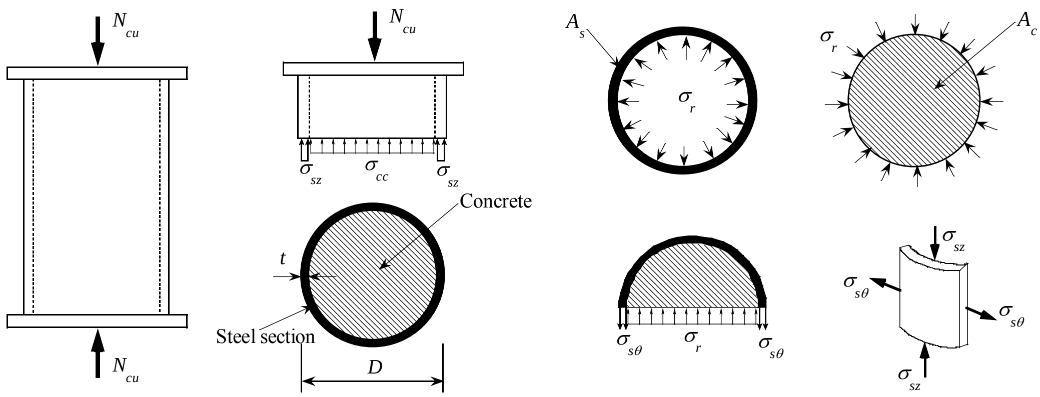

Figure 3 Stress state for a circular CFT column in a limited state

If σ_{cc} and σ_{sz} are determined, the axial strength of the CFT columns will be known. First, the strength of the confined concrete σ_{cc}, is assumed to be given by (Mander et al. 1988)

$$\sigma_{cc} = f_{cp} + k\sigma_r \tag{2.2}$$

where f_{cp} = the unconfined compressive strength of the filled concrete; k = the confinement coefficient = 4.1 (Richart et al. 1928).

According to Figure 3, the equilibrium of σ_r and $\sigma_{s\theta}$ gives

$$(D-2t)\cdot\sigma_r = -2t\cdot\sigma_{s\theta} \tag{2.3}$$

where D = the external diameter of the steel tube and t = the plate thickness of the steel tube.

Rewriting Eqn. 2.3, one obtains

$$\sigma_r = -\frac{2t}{(D-2t)}\cdot\sigma_{s\theta} \tag{2.4}$$

Substituting Eqn. 2.4 in Eqn. 2.2, one obtains

$$\sigma_{cc} = f_{cp} - \frac{2tk}{D-2t}\sigma_{s\theta} \tag{2.5}$$

Then, N_{cu} is expressed as

$$N_{cu} = A_c f_{cp} - \frac{2tk}{D-2t} A_c \sigma_{s\theta} + A_S \sigma_{sz} \tag{2.6}$$

Suppose the axial stress σ_{sz} and ring tension stress $\sigma_{s\theta}$ of the steel tube under ultimate load are expressed by

$$\sigma_{sz} = \alpha f_y, \quad \sigma_{s\theta} = \beta f_y \tag{2.7}$$

where α, β = coefficients; f_y = the yield strength of the steel tube.
Substituting Eqn. 2.7 in Eqn. 2.6 leads to

$$N_{cu} = A_c f_{cp} - \frac{2tk}{D-2t} A_c \beta f_y + A_S \alpha f_y \tag{2.8}$$

Eqn. 2.8 can be rewritten as

$$\begin{aligned} N_{cu} &= A_c f_{cp} - \frac{2tk}{D-2t} A_c \beta f_y + A_S \alpha f_y - A_S f_y + A_S f_y \\ &= A_c f_{cp} [1 - \frac{2k\beta t}{D-2t}\frac{f_y}{f_{cp}} + (\alpha - 1)\frac{A_S}{A_c}\frac{f_y}{f_{cp}}] + A_s f_y \end{aligned} \tag{2.9}$$

The ratio of the cross-sectional area of the steel tube to that of concrete is given by

$$\frac{A_s}{A_c} = \frac{\pi D^2/4 - \pi[(D-2t)/2]^2}{\pi[(D-2t)/2]^2} = \frac{4(D-t)t}{(D-2t)^2} \tag{2.10}$$

Substituting Eqn. 2.10 in Eqn. 2.9 produces

$$N_{cu} = A_c f_{cp}[1 + \frac{4(\alpha-1)(D/t-1) - 2k\beta(D/t-2)}{(D/t-2)^2}\frac{f_y}{f_{cp}}] + A_s f_y \tag{2.11}$$

Denoting

$$\eta_c = \frac{4(\alpha-1)(D/t-1) - 2k\beta(D/t-2)}{(D/t-2)^2}\frac{f_y}{f_{cp}} \tag{2.12}$$

Then, Eqn. 2.11 becomes

$$N_{cu} = A_c f_{cp}(1 + \eta_c) + A_s f_y \tag{2.13}$$

Here, η_c is called as the coefficient of confinement for the concrete core.
In order to determine the coefficient of confinement for the concrete core expressed in Eqn. 2.12, one needs to determine the values of α and β in Eqn. 2.7, which are found to be almost independent of material properties and dimensions of columns (Inai et al. 1995; Yamamoto et al. 2002). Based on the assumption that steel stresses

at the limit state given by Eqn. 2.7 satisfies the Von Mises yield criterion, the relationship between α and β is given by

$$\alpha^2 - \alpha\beta + \beta^2 = 1 \tag{2.14}$$

Considering the result that ring tensile stress is obtained as approximately 20% of the yield stress of the steel tube by a large number of experiments (e.g., Sato 1993; Suzuki et al. 1997), the values of coefficients α and β that satisfy both the experimental results and Eqn. 2.14, are taken to be 0.88 and -0.21, respectively, in the present study.

Substituting the values of α, β, and k in Eqn. 2.12, the coefficient of confinement for concrete core η_c becomes

$$\eta_c = \frac{(1.24D/t - 2.96)}{(D/t-2)^2}\frac{f_y}{f_{cp}} \tag{2.15}$$

Simplifying Eqn. 2.15, an approximating formula is given as

$$\eta_c = 1.25\frac{t}{D}\frac{f_y}{f_{cp}} \tag{2.16}$$

The comparison of Eqn. 2.15 and Eqn. 2.16 were illustrated in Figure 4 (a, b, and c) for f_y/f_{cp} = 5, 10, and 20, respectively. One can clearly see that the coefficient of confinement for concrete core η_c obtained from Eqn. 2.16 is in good agreement with that calculated by using Eqn. 2.15.

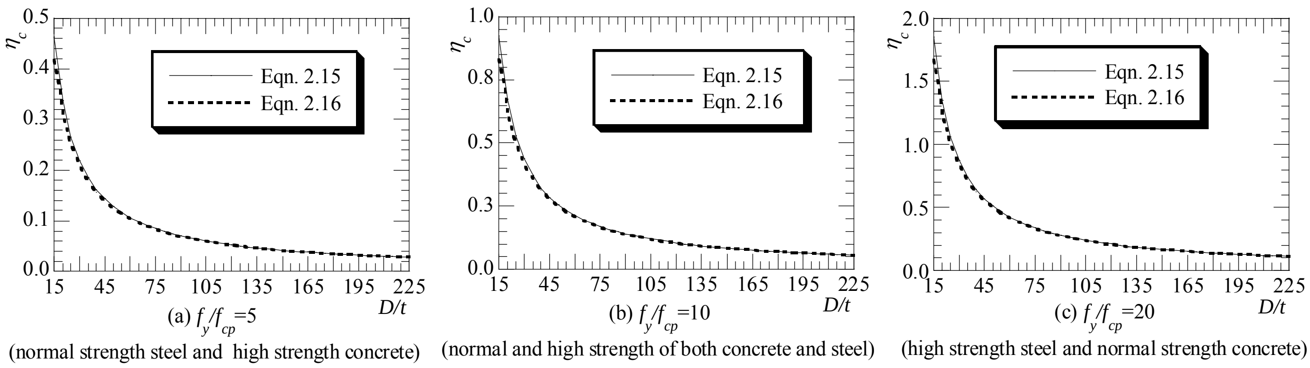

Figure 4 Comparisons between Eqn. 2.15 and Eqn. 2.16

From Eqn. 2.16, it is not difficult to understand that the coefficient is dependent on the column D/t ratio, the yield stress of the steel tube (f_y), and the unconfined compressive strength of the concrete core (f_{cp}). Because the coefficient of confinement for the concrete core is proportional to the strength ratio f_y/f_{cp} between the steel and concrete, if D/t is fixed, simultaneously increasing of f_y and f_{cp} will not increase the confinement effect.

The variations of the confinement coefficient (η_c) with respect to D/t, f_y and f_{cp} are depicted in Figure 5(a, b and c), respectively. As can be seen from Figure 5a, there is a sharp decrease of η_c when the diameter-to-thickness (D/t) ratio is small and that η_c tends to be moderate when $D/t > 60$. This is similar to O'Shea and Bridge's (1997, 2000) conclusions that a steel tube with a D/t ratio being greater than 55 provides insignificant confinement for the concrete core. From Figure 5a and 5b, one can see that the η_c decreases when f_{cp} increases and increases when f_y increases, which means that CFT columns with high-strength concrete and low-strength steel tubes do not provide good confinement efficiency for the CFT columns.

The unconfined compressive strength of filled concrete f_{cp} in Eqn. 2.13 and Eqn. 2.16 is estimated from the following equation (Sakino et al. 2004)

$$f_{cp} = \gamma_c f_c' = 1.67 D_c^{-0.112} f_c' \tag{2.17}$$

where f_c' = the unconfined compressive cylinder (with a diameter of 100 mm and height of 200 mm) strength of the concrete; D_c = the diameter of the concrete core; and the γ_c is a strength reduction factor introduced to take the scale effect into consideration.

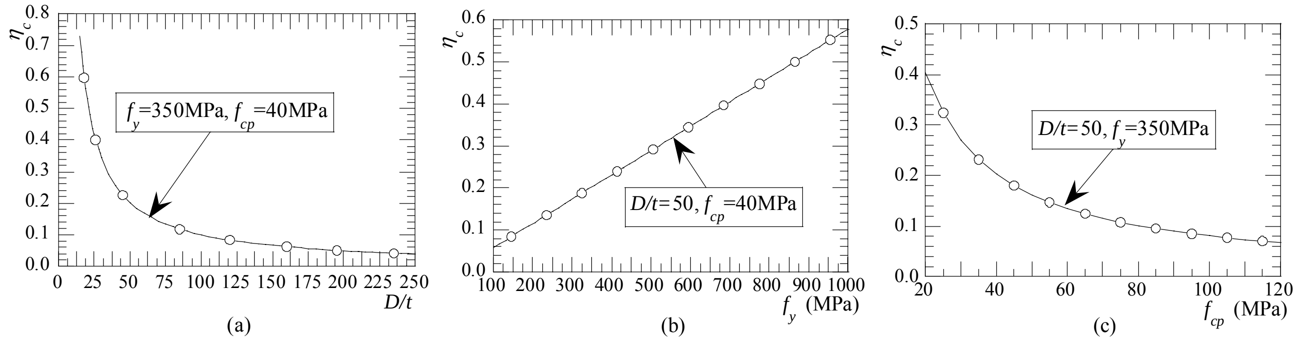

Figure 5 η_c versus D/t, f_{cp} and f_y

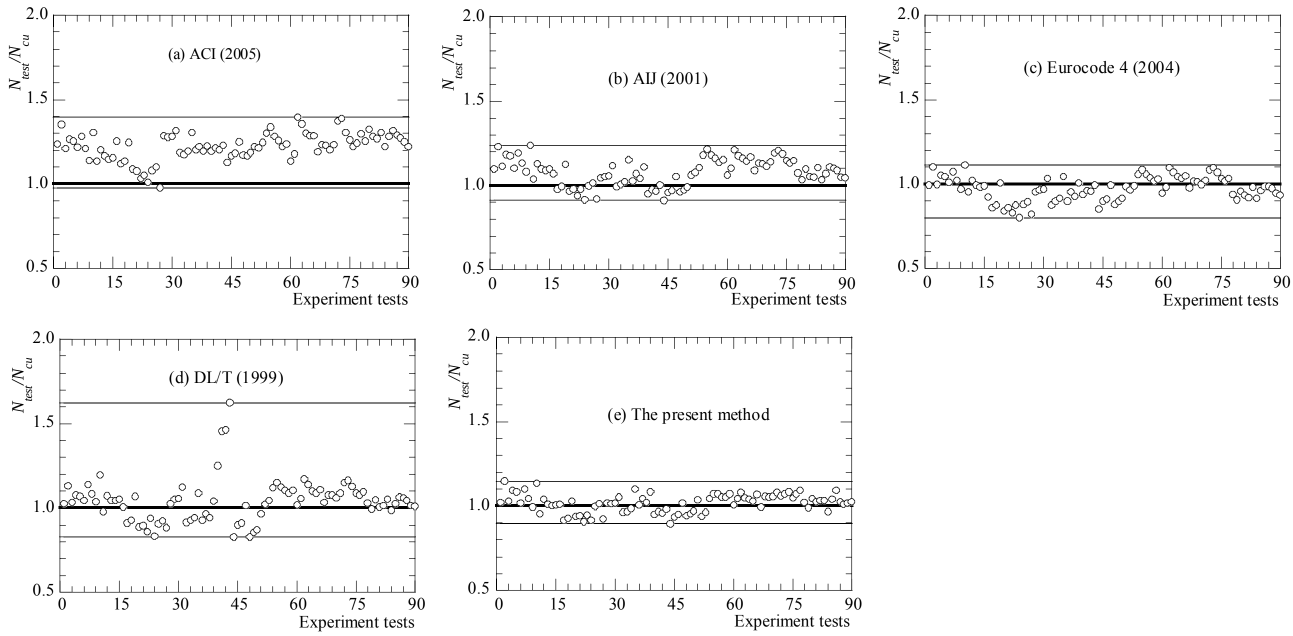

Figure 6 Comparison of experimental results with results predicted section capacities

3. VERIFICATION OF THE PROPOSED FORMULA

Recent experimental investigations of circular CFT short columns conducted by O'Shea and Bridge (2000) (15 tests), Yamamoto et al. (2002) (13 tests), Sakino et al. (2004) (36 tests) and Han et al. (2005) (26 tests) were used to verify the proposed formula in this study. The main parameters varied in these tests are: (1) diameter-to-thickness ratio: from 16 and 220; (2) the compressive strength of concrete: from 20 to 100MPa; and (3) the yield strength of the steel tube: from 185 to 853 MPa.

For overall comparisons, the mean, standard deviation, maximum, minimum, and the differences between maximum and minimum of the ratio of N_{test}/N_{cu} for the different design methods are shown in Table 3.1, in which N_{cu} = predicted section capacities using the different methods and N_{tes}= test results. The N_{test}/N_{cu} ratio versus the experiment tests is depicted in Figure 6.

As can be seen from Table 3.1 and Figure 6, ACI (2005) gives a sectional capacity about 22.4% lower than the experimental results, mainly because the composite action between the steel tube and the concrete core was not considered. AIJ (2001) gives a sectional capacity about 8% lower than the measured ultimate strengths, and the N_{test}/N_{cu} ratio ranges from 0.914 to 1.239 with a standard deviation of 8%. Although DL/T (1999) gives section capacity about 4.3% lower than the measured ultimate strengths, the standard deviation of the N_{test}/N_{cu} ratio is the largest among all the methods and the ratio ranges from 0.828 to 1.625, which means the uncertainty in values predicted using DL/T (1999) is large. Eurocode 4 (2004) gives unconservative results with a mean and standard deviation of the N_{test}/N_{cu} ratio of 0.974 and 0.071, respectively. Overall, the proposed method gives the best representation of the ultimate strength of circular CFT stub columns with a mean of 1.018 and a standard deviation of 0.054.

Table 3.1 Comparison between predicted section capacities (N_{cu}) and test results (N_{test})

	N_{test}/N_{cu}				
	ACI (2005)	AIJ (2001)	Eurocode (2004)	DL/T (1999)	The present method
Mean	1.224	1.080	0.974	1.043	1.018
Standard deviation	0.081	0.080	0.071	0.126	0.054
Maximum	1.396	1.239	1.111	1.625	1.147
Minimum	0.979	0.914	0.799	0.828	0.895
Maximum-Minimum	0.417	0.325	0.312	0.797	0.252

4. CONCLUSIONS

1. An accurate formula for predicting the axial capacity of circular CFT stub columns with normal- and high-strength steel and concrete is proposed. The scale effect on the strength of the filled concrete and the enhancement of CFT columns due to the composite action between steel tube and concrete core are taken into account in the proposed formula.
2. A sharp decrease of concrete confinement is found when the diameter-to-thickness ratio is small, and it tends to be moderate when the diameter-to-thickness ratio is greater than 60. The concrete confinement increases linearly when the yield stress of the steel tube increases, and it decreases as the unconfined compressive strength of the concrete core increases, respectively.
3. ACI (2005) give a sectional capacity about 22.4% lower than the experimental results, and AIJ (2001) and DL/T (1999) give a sectional capacities about 8% and 4.3% lower than the experimental results, respectively, while the Erurocode 4 (2004) predicts unconservative results with 2.6% higher than the results obtained in the tests.
4. The values predicted using the present formula are in good agreement with the experimental results for circular CFT stub columns not only within a large range of diameter-to-thickness ratios but also with normal-strength of concrete and steel tubes and high-strength of concrete and steel tubes.

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