

## **Experimental and Theoretical Comparative Study of Circular Steel Tubular Columns Filled with Self-Compacting Concrete under Axial Concentric Loading**

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### **Abstract :**

*This study presents an experimental and theoretical study on the behavior of circular, concrete-filled, steel tube (CFT) columns filled with self-compacting concrete (SCC) concentrically loaded in compression to failure. Specimens were tested to investigate the ultimate capacity and the load–deformation behavior of the columns. The behavior of these columns in confinement was discussed. The test results for acceptance characteristics of self-compacting concrete (SCC) such as slump flow; V-funnel and L-Box are presented. The experimental study was used to compare with theoretical results based on three different building codes. The codes used were the Eurocode 4 - EN 1994-1-1:2004 (EC4), ANSI/AISC 360:2010 and AIJ. The test specimens were of a length-to-diameter ratio (L / D) of 2.5, 6.25 and 9.375 respectively. The internal core is self-compacting concrete had nominal unconfined cylinder strength of 30 MPa. In general, the code provisions were of different accuracy in the prediction of column capacity. The AISC (1999) was the most conservative, while Eurocode 4 presented the values closest to the experimental results. The slenderness ratio was critical in determining the formation of a local buckle, and those slenderest columns only attained 85.7% of their section capacities.*

**دراسة مقارنة عملية و نظرية للاعمدة الفولاذية الدائرية المملوءة بخرسانة ذاتية الرص  
تحت تأثير احمال محورية**

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**الخلاصة :**

في هذه البحث تم اجراء دراسة عملية و نظرية لتصرف الاعمدة المركبة الدائرية المملوءة من انبوب فولاذي و لب من الخرسانة ذاتية الرص و محملة بحمل أنضغاط محوري حتى مرحلة الفشل. تم فحص عينات لدراسة التحمل الاقصى و تصرف حمل - تشوه للاعمدة. تم مناقشة تأثير اجهادات الحصر على سلوك الاعمدة. لتأكيد تصرف الخرسانة ذاتية الرص تم اجراء الفحوصات الخاصة بهكذا نوع من الخرسانة مثل فحص الجريان فحص الصندوق الزاوي و القمع الهرمي. تم استخدام احكام ثلاث من مدونات الابنية و ذلك لحساب قدرة تحمل الاعمدة المركبة من انابيب فولاذية مملوءة

بالخرسانة و تحت تأثير احمال محورية و مقارنتها مع فحوصات مختبرية. المدونات المستخدمة هي: الاوروبية و الامريكية و اليابانية. العينات المفحوصة كانت بنسب طول اقطر: 2.5 و 6.25 و 9.375. اللب الداخلي للاعمدة مكون من الخرسانة ذاتية الرص و بمقاومة انضغاط  $30 \text{ MPa}$ . بصورة عامة، فان المدونات كانت ذات دقة متفاوتة في تخمين مقدار التحمل للاعمدة. المدونة الامريكية (1999) كانت الاكثر تحفظاً بينما المدونة الاوروبية هي الاكثر قرباً للنتائج العملية. كان لنسبة النحافة اثرأ مهماً بشكل الفشل و كانت الاعمدة الانحف قد أظهرت 85.7% من مقاومة المقطع.

## 1. Introduction :

Steel members have the advantages of high tensile strength and ductility, while concrete members have the advantages of high compressive strength and stiffness. Composite members combine steel and concrete, resulting in a member that has the beneficial qualities of both materials. The advantage of concrete filled circular steel tubular (CFT) columns is that the steel tube serves as a form for casting the concrete, which reduces construction cost. Also, no other reinforcement is needed since the tube acts as longitudinal and lateral reinforcement for the concrete core. In addition, the placement of longitudinal steel at the perimeter of the section is the most efficient use of the material since it provides the highest contribution of the steel to the section moment of inertia and flexural capacity. The continuous confinement provided to the concrete core by the steel tube enhances the core's strength and ductility. The concrete core delays local buckling of the steel tube by preventing inward buckling, while the steel tube prevents the concrete from spalling <sup>[1]</sup>.

Olivera et al. <sup>[2]</sup> studied CFT columns with concrete core of compressive strengths ranged from 30 to 120 MPa and of length/diameter ratios ranged from 3 to 10. The experimental values of the columns' ultimate load were compared to the predictions of four code provisions: the Brazilian Code NBR 8800 <sup>[3]</sup>, Eurocode 4 <sup>[4]</sup>, AINSI/AISC 360 <sup>[5]</sup>, and CAN/CSA S16-0 <sup>[6]</sup>. According to the results, the load capacity of the composite columns increased with increasing concrete strength and decreased with increasing length/diameter ratio. In general, the code provisions were highly accurate in the prediction of column capacity. Among them, the Brazilian Code was the most conservative, while Eurocode 4 presented the values closest to the experimental results.

Kilpatrick et al. <sup>[7, 8]</sup> examined the applicability of Eurocode 4 <sup>[4]</sup> for the design of CFTs which use high strength concrete and compared 146 columns from six different investigations with Eurocode 4. The concrete strength of the columns ranged from 23 to 103 MPa. The mean ratio of measured / predicted column strength was 1.10 with a standard deviation of 0.13. The Eurocode safely predicted the failure load in 73% of the columns analyzed.

The behavior of circular concrete-filled steel tubes (CFT) with various concrete strengths under axial load was also presented by Giakoumelis and Lam <sup>[9]</sup>. The effects of steel tube thickness, the bond strength between the concrete and the steel tube and the confinement of concrete were examined. Measured column strengths were compared with the results predicted using Eurocode 4 <sup>[4]</sup>, Australian standards <sup>[10, 11]</sup> and American codes <sup>[12]</sup>. All three codes predicted lower values than that measured during the experiments. Eurocode 4 gave the best estimation for both CFT with normal and high-strength concrete. It was also found that the effect of concrete shrinkage was critical for high strength concrete and negligible for normal strength concrete.

Although there have been a large number of studies on CFT columns with normal and high-strength concrete, there has been relatively little research on CFT columns with self-compacting concrete (SCC) <sup>[13]</sup>. SCC possesses high workability, whereby the concrete can flow under its own weight and fill the formwork completely. Due to the rheological properties of SCC, the expense of vibration is eliminated whilst still obtaining a good compaction.

Furthermore, advantages of SCC include a reduction of noise level in manufacturing plants and a reduction in construction time and labor costs. Therefore, there is a good potential for using CFT columns with SCC in structures. Han and Yao <sup>[13]</sup> indicated that the load carrying capacity and failure modes of CFT columns filled with SCC and with NC were very similar if the concrete strength is close.

## 2 . Self-Compacting Concrete Tests :

Numerous efforts have been explored for new testing methods on SCC in the past decade. There are several organizations that collect the work in this area. ACI Committee 237 Specification and Guidelines for Self-Compacting Concrete <sup>[14]</sup>, EFNARC (2002) <sup>[15]</sup> and EFNARC (2005) <sup>[16]</sup> are good examples.

A mix design of the SCC is given in **Table 1**. The maximum coarse aggregate size was 19 mm.

**Table 1 SCC mix design.**

Cement (kg /m <sup>3</sup> )	Sand (kg /m <sup>3</sup> )	Gravel (kg /m <sup>3</sup> )	Water (kg /m <sup>3</sup> )	Super- plasticier
470	868	841	155	1.4%

To assure SCC properties, four typical tests were used here and they are:

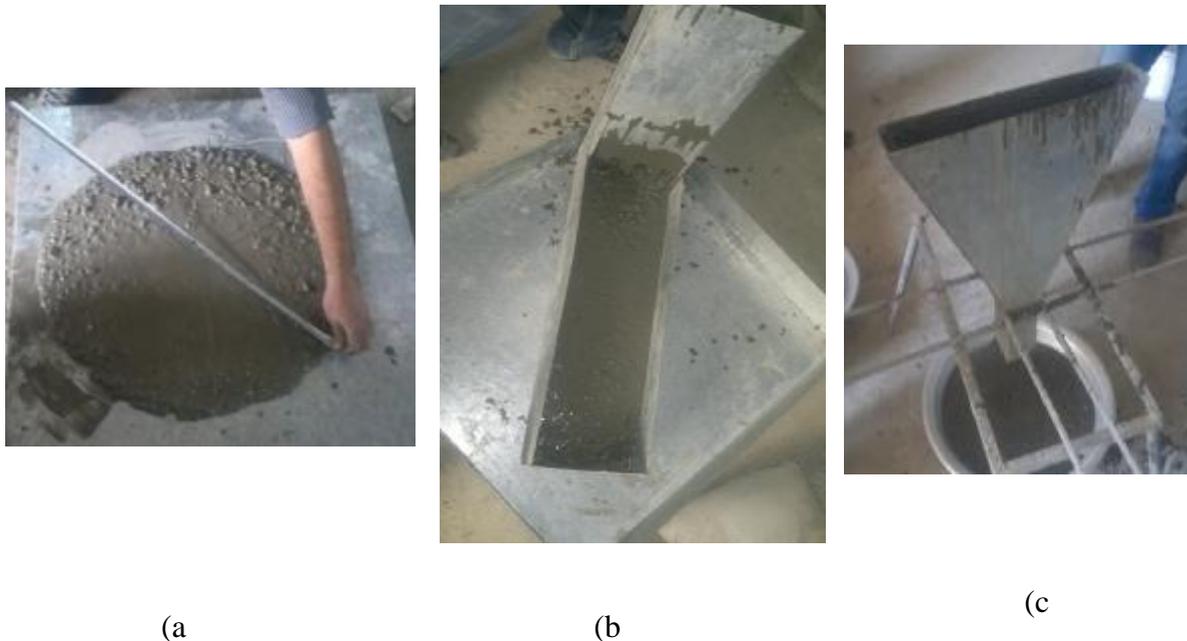
1. The Slump Flow Test: A test method for evaluating the flowability of SCC (**Fig. 1 (a)**). The average of the diameter of flowing is 730 mm which is within the limits set by EFNARC <sup>[15]</sup>.
2. T50 Test: A test method for evaluating the rate of filling of SCC, where the 500 mm flow reach time is measured in the slump flow test above. SCC should give T50 = (2 – 5) seconds <sup>[15]</sup>. Here, the time was 4 seconds.
3. L-Box Test: A test method for evaluating the passing ability of SCC (**Fig. 1 (b)**). The specified requisite is the ratio between the heights of the concrete at each end or blocking ratio to be (0.8 – 1.0). Here the ratio was 1.0 which is satisfying the requirements <sup>[15]</sup>.
4. V-Funnel Test: A test method for evaluating both the filling ability and the material segregation resistance of SCC (**Fig. 1 (c)**). The flow time for all of the concrete to exit the funnel is recoded as a measure of filling ability. The flow time was 7 seconds which is less than 10 seconds. To measure segregation resistance, the V-funnel is refilled with concrete and allowed to sit for 5 minutes. The door is again opened and the flow time is recorded. The greater the increase in flow time after the concrete has remained at rest for five minutes, the greater will be the concrete's susceptibility to segregation. Further, non-uniform flow of concrete from the funnel suggests a lack of segregation resistance <sup>[16]</sup>. Here 11 seconds in the second phase and the flowing is uniform which prove that segregation is not expected to happen.

### 3 . Composite Columns Test :

A total of six test specimens were constructed and tested under concentric axial compression loads. All the specimens are of 160 mm diameter ( $D$ ). Two of specimens were of length 400 mm ( $L$ ) to reduce the end effects and to ensure that the specimens would be stub columns with minimum effect from slenderness. Other two are of length 1000 mm that to study the effect of medium slenderness and the other two are of 1500 mm to see the effect of high slenderness. Each tube was welded to a square (200×200) mm, steel base plate of 5 mm thickness at the bottom. The SCC for filling in the steel tube columns were mixed first, and then the CFT columns were cast. Meanwhile, the corresponding SCC specimens of nine 150 mm cubes were cast for concrete strength tests.

The specimens are presented in Table 2, where  $D$  is the outside diameter of the circular steel tubes;  $t$  is the wall thickness of steel tube;  $L$  is the length of the specimen and is chosen to be variable. In addition to summary of the specimens information are given in **Table 1**, the experimental ultimate loads are included.

For the specimens where the load is applied to the entire section, another square steel cover plate of 5 mm in thickness was placed to the top surface of the steel tube. This was done to ensure that the load was applied evenly across the cross-section and simultaneously to the steel and concrete core. To study the effect of rigidity of the cover plate, very rigid plate was also placed. Even numbered specimens were loaded through very rigid plate and odd numbered specimens were loaded using 5 mm end plate.



**Fig.( 1) Self-compacting concrete tests: (a) Slump flow test, (b) L-box test, (c) V-funnel test.**

Then the column specimens were placed directly into the testing machine for compression tests, and it restrained against lateral movement only on both top and bottom of the columns due to friction effect. A typical column test layout and instrumentation location is shown in **Figures. 2** and 3. The concentric loads were applied on the specimens through the steel-bearing plate which will work as a simple support. Several strain gauges were used for two CFT specimens to measure the variation of strains at the mid-height of the specimen. Six electrical strain gauges were placed on the exterior surfaces of both the short ( $L / D = 2.5$ ) and long ( $L / D = 9.375$ ) columns to measure the vertical deformations and the perimeter expansion of the steel tubes in the mid-height region at symmetric locations, as shown in **Figure. 3**. Dial gauges were used to measure the axial deformation.

**Table 2 Summary of the specimen information.**

Specimen No.	$D \times t \times L$ (mm)	$L / D$	$F_y$ (MPa)	$f_{cu}$ (MPa)	$P_e$ (kN)
1	160 × 2.8 × 400	2.500	368	30	1370
2					1420
3	160 × 2.8 × 1000	6.250			1300
4					1370
5	160 × 2.8 × 1500	9.375			1210
6					1260

The experimental study was to determine not only the maximum load-bearing capacity of the composite specimens subjected to axially local compression, but also to investigate the failure pattern up to the ultimate load. All the tests were performed on a 2500 kN capacity testing machine.

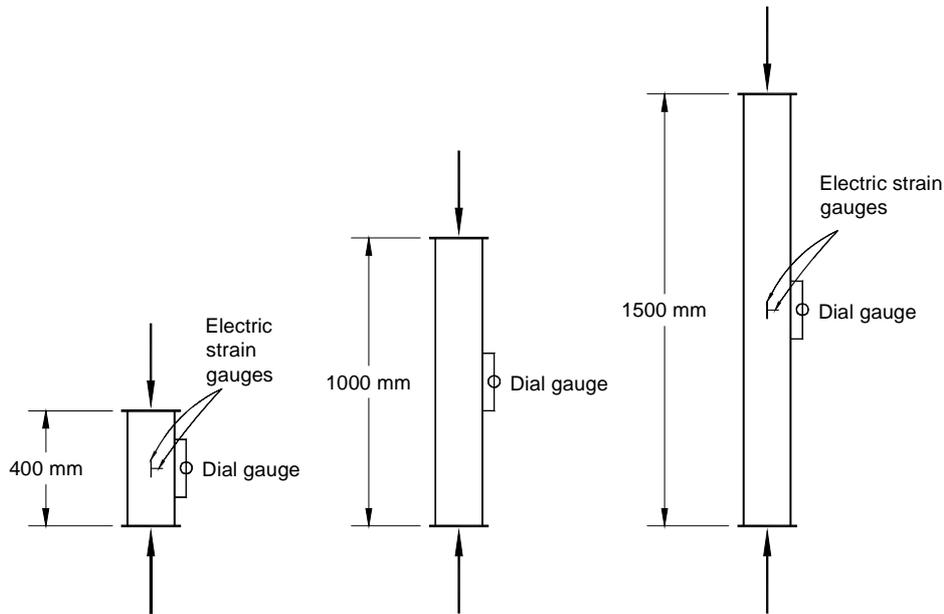
The specimens were loaded continuously until failure. A load interval of less than one-tenth of the estimated carrying load capacity was used. The progress of deformation, the mode of failure and the maximum load taken by the specimens were recorded.

It was found that the tested CFT columns under compression generally exhibited in a ductile manner, and the longitudinal force carried by the steel tube increased with the increase of the top endplate rigidity. This may due to the higher ductility of steel tube which allows redistributing stresses on the all composite section.

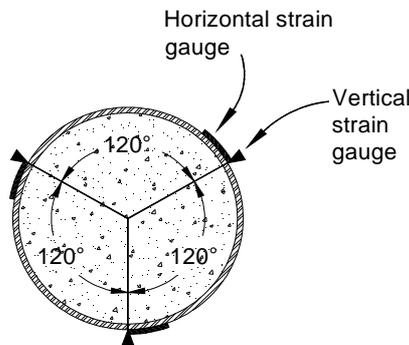
It can be found that, the deformation of the top part of steel tube becomes more obvious for the specimens with thicker endplate. The buckle of the steel tube focused on the position near the top endplate, as shown in **Figure( 4) .**

As can be seen from **Figure .( 5)**, the larger the slenderness ratio gives the smaller value of ultimate load. The shortest columns have attained their section capacities, whilst those slenderest columns only attained 85.7% of their section capacities. It is thus expected that slenderness reduction factors should be applied in designing slender CFT columns.

The failure mode of the specimens was a function of the  $L/D$  ratio. The short columns ( $L/D = 2.5$ ) failed due to the crushing of the concrete core, aggravated by local buckling of the steel tube after having reached the yielding stress of the steel.



**Fig.( 2) Test layout.**



**Fig. (3) Strain gauges distribution.**

Due to slenderness effect, the columns of  $L/D = 2.5$  reached a strain of about 6%, as shown in **Figure( 6)**, while specimens with  $L/D = 6.25$  and  $9.375$ , presented global instability **Figure.( 7)** and the failure occurred at axial strain lower than specimens with lower  $L/D$  ratio. As in **Figure.( 6)** the strain reached for  $L/D = 9.375$  is approximately 3% which is half the value for short column.



**Fig. (4) Failure of the short column.**

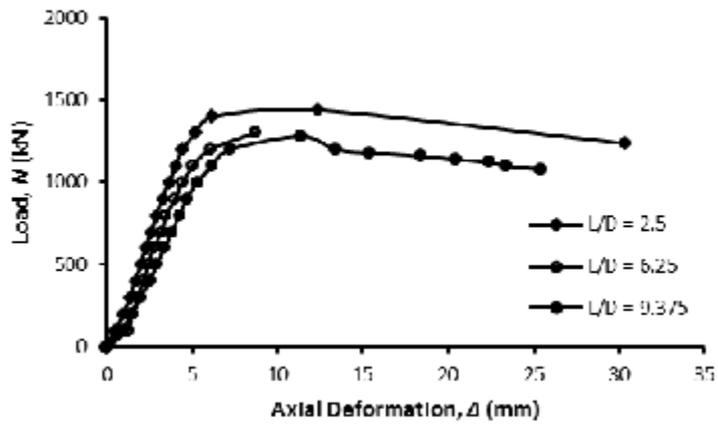


Fig.( 5) Load-deformation behavior.

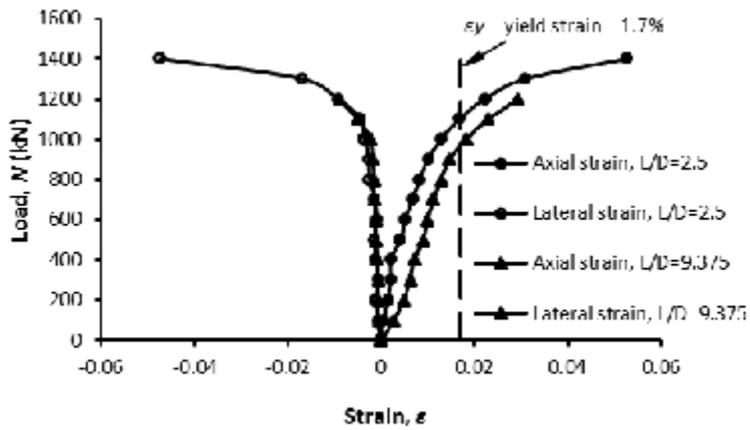


Fig.( 6) Load-strain relationship.



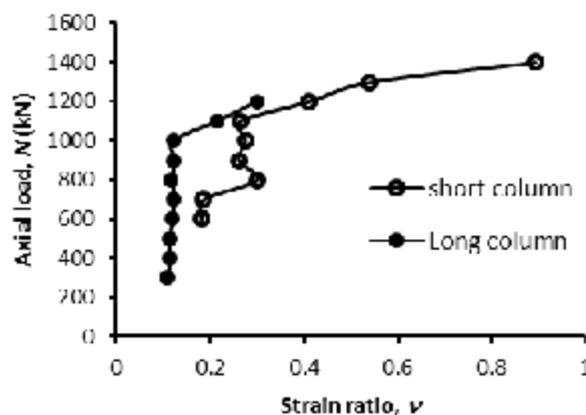
Fig.( 7) Failure of the long column.

It was also observed that the load-strain curve changed after the specimens reached the yielding strain of the steel tube (1.7%). All strain gauges placed to measure axial strain obtained values higher than 1.7% at the peak load. This seems to indicate the beginning of the local buckling process for the short columns ( $L/D = 2.5$ ) and the global instability for the columns with  $L/D = 9.375$ .

The specimens with  $L/D = 9.375$  exhibited insufficient lateral strain for mobilizing the confinement effect. This was verified by the three strain gauges placed outside around the column. The lateral strain measured was about 1.5% for the columns with  $L/D = 9.375$  and 5% for the short columns at the ultimate load.

From **Figure.( 8)** it can be noted that in the early stages of loading that minor changes in the strain ratio (lateral to axial) recorded, but a dramatic change occurred after strain yield reached. This may due to that in the early stages of loading, Poisson’s ratio for concrete is lower than that for steel, and the steel tube has no restraining effect on the concrete core. As the longitudinal strain increases, Poisson’s ratio of concrete which is 0.15–0.2 in the elastic range increases to 0.5 in the inelastic range <sup>[17]</sup>. Therefore, the lateral expansion of unconfined concrete gradually becomes greater than that of steel.

A radial pressure develops at the steel–concrete interface thereby restraining the concrete core and setting up a hoop tension in the tube. At this stage, the concrete core is stressed triaxially and the steel tube biaxially, so that there is a transfer of load from the tube to the core, as the tube cannot sustain the yield stress longitudinally in the presence of a hoop tension. The load corresponding to this mode of failure can be considerably greater than the sum of the steel and concrete, but shear failure may intervene before the load transfer is complete <sup>[17]</sup>. **Figure. (8)** is not showing the first points because of accuracy problems.



**Fig.(8) Load-strain ratio relationship.**

## 4 . Predicting the Compressive Strength of Filled Steel Columns for Design :

Different approaches giving significant discrepancies in results are presently being used in different places of the world for the calculation of the ultimate strength or squash load of composite columns, and generally involve the summation of factored strengths of the components forming the composite section i.e. the fill material and the steel tube.

Different formulae recommended by national or regional codes, namely the Architectural Institute of Japan SRC-2001 [17, 18], the European code EC4 [4] and the American Institute of Steel Commission AISC LRFD [19, 20]. In all the design calculations, the resistance factors and material partial factors are set to one.

For completeness, a brief review of the determination of the axial capacity of circular CFT columns using the methods described in the codes is presented as follows.

### 4.1 AISC LRFD Composite Column Design (1999)

The AISC composite column design provisions are subject to the following limitations [19]:

- The area of the steel section must be at least 4 percent of the composite cross section.
- The concrete strength must be between 3 and 8 ksi (21 and 56 MPa) for normal weight concrete and at least 4 ksi (28 MPa) for lightweight concrete.
- The maximum yield stress of either structural steel or reinforcing bar must not exceed 60 ksi (420 MPa) for calculations.
- The minimum thickness for circular concrete-filled tubes with an outside diameter of  $D$  is  $D\sqrt{F_y/8E}$ , where  $F_y$  is the yield strength and  $E$  is the modulus of elasticity of steel tube.

The AISC composite column design procedure is similar to the steel column design procedure, except that it uses modified properties calculated from the composite cross-section instead of the steel section properties. A modified yield stress,  $F_{my}$ , modified elastic modulus,  $E_m$ , and modified radius of gyration,  $r_m$  are required to design composite column. These parameters are given by Eqs. 1 to 3.

$$F_{my} = F_y + c_1 F_{yr} \left( \frac{A_r}{A_s} \right) + c_2 f_c \left( \frac{A_c}{A_s} \right) \quad (1)$$

$$E_m = E_s + c_3 E_c \left( \frac{A_c}{A_s} \right) \quad (2)$$

$$r_m = \max. (r_{steel}, 0.3h_1) \quad (3)$$

where,  $A_c$  = area of concrete (mm<sup>2</sup>),  $A_r$  = area of longitudinal reinforcing bars (mm<sup>2</sup>),  $A_s$  = area of steel (mm<sup>2</sup>),  $E_s$  = modulus of elasticity of steel (MPa),  $E_c$  = modulus of elasticity of

concrete (MPa), which equals  $0.043w^{1.5}\sqrt{f'_c}$  or  $4700\sqrt{f'_c}$  where  $w$  is the unit weight of the concrete in  $\text{kg} / \text{m}^3$  and  $f'_c$  in MPa,  $F_y$  = specified minimum yield stress of the steel shape, pipe or tube (MPa),  $F_{yr}$  = specified minimum yield stress of the longitudinal reinforcing bars (MPa),  $f'_c$  = specified compressive cylinder strength of concrete (MPa), and  $h_1$  = overall thickness of entire composite cross-section in the plane of buckling (mm).

Depending on the type of composite cross-section, the coefficients  $c_1$ ,  $c_2$ , and  $c_3$  have different values. For concrete filled tubes and pipes these coefficients are  $c_1 = 1.0$ ,  $c_2 = 0.85$  and  $c_3 = 0.4$ .

The modified column slenderness parameter,  $\lambda_m$  is defined as

$$\lambda_m = \frac{kl}{r_m \pi} \sqrt{\frac{F_{my}}{E_m}} \quad (4)$$

where  $k$  = effective length factor,  $l$  = laterally unbraced length of the member, mm.

As for steel only sections, based on this slenderness, the critical stress for  $\lambda_m \leq 1.5$  is calculated by

$$F_{cr} = (0.658^{\lambda_m}) F_y \quad (5)$$

and for  $\lambda_m > 1.5$  by

$$F_{cr} = \frac{0.877}{\lambda_m^2} F_{my} \quad (6)$$

The nominal strength of the column  $P_n$  is

$$P_n = A_s F_{cr} \quad (7)$$

and design equation is

$$P_u = \phi_c P_n \quad (8)$$

where  $\phi_c$  is the resistance factor which equals 0.75.

## 4.2 LRFD Composite Steel Column Design (2010)

The proposed AISC 2010 Unified Specification <sup>[20]</sup> contains significant changes in the design of composite columns. In this section, these revisions are introduced and compared with the provisions of the 1999 AISC LRFD specification that was discussed in the previous section. Composite column design in the 2010 Specification is subject to the following limitations <sup>[20]</sup>:

- The cross sectional area of the steel must comprise at least 1% of the composite cross section. This limit was 4% in the 1999 AISC LRFD.
- The concrete strength must be between 3 and 10 ksi (21 and 70 MPa) for normal weight concrete and between 3 and 6 ksi (21 and 42 MPa) for lightweight concrete. In the previous specification, the upper limit for normal weight concrete was 8 ksi (56 MPa). The limits for lightweight have been changed from a minimum of 4 ksi (28 MPa) to a range of 3 to 6 ksi (21 and 42 MPa) from the 1999 AISC LRFD.
- The maximum  $D / t$  for circular concrete-filled tubes shall be  $0.15 E / F_y$ . This has been liberalized from the 1999 AISC LRFD limit of  $\sqrt{8E/F_y}$ .

The 2010 AISC Composite column design method has different equations for cross-sectional strength depending on whether columns are encased composite columns and filled composite columns. The cross-sectional strength is based on the plastic capacity of the section. For filled composite columns,

$$P_0 = A_s F_y + A_r F_{yr} + C_2 f'_c A_c \quad (9)$$

$$EI_{eff} = E_s I_s + 0.5 E_s I_r + C_3 E_c I_c \quad (10)$$

$$C_3 = 0.6 + 2 \left( \frac{A_s}{A_c + A_s} \right) \leq 0.9 \quad (11)$$

where  $C_2 = 0.85$  for rectangular sections and  $0.95$  for circular sections,  $A_c =$  area of concrete,  $\text{mm}^2$ ,  $A_r =$  area of continuous reinforcing bars,  $\text{mm}^2$ ,  $A_s =$  area of steel section,  $\text{mm}^2$ ,  $E_c =$  modulus of elasticity of concrete, MPa,  $= 0.043w^{1.5}\sqrt{f'_c}$  or  $4700\sqrt{f'_c}$  where  $w$  is the unit weight of the concrete in  $\text{kg} / \text{m}^3$  and  $f'_c$  in MPa,  $E_s =$  modulus of elasticity of steel, MPa,  $EI_{eff} =$  effective rigidity of composite section,  $\text{N}\cdot\text{mm}^2$ ,  $f'_c =$  specified minimum concrete compressive strength, MPa,  $F_y =$  yield stress of the steel section, MPa,  $F_{yr} =$  specified minimum yield stress of reinforcing bars, MPa,  $I_c =$  moment of inertia of the concrete section,  $\text{mm}^4$ ,  $I_s =$  moment of inertia of the steel shape,  $\text{mm}^4$ ,  $I_r =$  moment of inertia of reinforcing bars,  $\text{mm}^4$ , and  $w_c =$  weight of concrete per unit volume.

The design strength is given as:

$$\phi_c P_n \geq P_u \quad (12)$$

where the resistance factor is:

$$\varphi_c = 0.75 \quad (13)$$

and the nominal strength  $P_n$  is given by:

$$P_n = \Lambda P_0 \quad (14)$$

Based on the column slenderness, when  $\alpha \leq 1.5$ ,

$$\Lambda = 0.658\alpha^2 \quad (15)$$

and when  $\alpha \geq 1.5$  is

$$\Lambda = \frac{0.877}{\alpha^2} \quad (16)$$

where  $\alpha = \sqrt{P_0/P_e}$ ,  $P_e = \pi^2(EI)_{eff}/(kl)^2$  and  $\Lambda$  is a factor calculates for slenderness effect.

### 4.3 Eurocode 4

The Eurocode 4 column design assumes that concrete and steel interact fully with each other until failure<sup>[4]</sup>. Design by the Eurocode method uses the full plastic axial and moment capacity of the cross-section and then reduces those values based on the column slenderness ratio and other factors. The Eurocode composite design considers all material properties of the cross-section, including partial safety factors for the different materials. The Eurocode uses partial safety factors to reduce steel yield stress, concrete compressive strength, and yield stress of reinforcing bar, while AISC uses a single resistance factor. This is one of the reasons why the Eurocode procedures are more complex than the AISC composite column design ones.

The composite column is required to meet the following limitations:

- The composite column is doubly symmetric and of uniform cross-section over the whole column length.
- The slenderness ratio parameter of the column,  $\lambda$ , is less than 2.0.
- The minimum requirement of the longitudinal reinforcement ratio is 0.3%.

For compression members, local buckling of the steel is checked first. Each type of cross-section must meet certain minimum depth-to-thickness ratios<sup>[4]</sup>. For circular sections,

$$\frac{D}{t} \leq S \cdot \epsilon^2 \quad (17)$$

where  $S$  is a factor depends on the yielding strength of steel. The term  $\epsilon$  is function of the yield strength of the steel,

$$\varepsilon = \sqrt{\frac{235}{F_y}} \quad (18)$$

where  $F_y$  in MPa.

The plastic resistance of cross-sections subjected to axial loads is given by Eq. 19 below. This equation combines the resistance of the structural steel, the concrete and the reinforcement. The confinement effect is not taken into account when the slenderness ratio of the column  $\lambda$  is greater than 0.5 and the eccentricity of loading,  $e$ , is greater than  $D/10$ , where  $D$  is the outside diameter of the steel tube. Thus, the strength equation for concrete filled circular cross-sections is:

$$P_{pt} = A_s \frac{F_y}{\gamma_s} \eta_2 + A_c \frac{f_c'}{\gamma_c} \left( 1 + \eta_1 \frac{t}{D} \frac{F_y}{f_c'} \right) + A_r \frac{F_{yr}}{\gamma_r} \quad (19)$$

where  $A_s$ ,  $A_c$  and  $A_r$  are the cross-sectional area of the structural steel, the concrete and the reinforcement respectively.  $\gamma_s$  is partial safety factor for the structural steel,  $\gamma_c$  is partial safety factor for the concrete, and  $\gamma_r$  is partial safety factor for the reinforcing steel. The values of  $\gamma_s$ ,  $\gamma_c$  and  $\gamma_r$  are taken as 1.1, 1.5 and 1.15 respectively.

The coefficients  $\eta_1$  and  $\eta_2$  account for the confinement effect. The strength of the concrete is increased by  $\eta_1$  because concrete has a higher strength when a triaxial state of stress occurs. The strength of the steel tube is decreased by  $\eta_2$  because the effective yield stress of the steel is reduced by the hoop stresses. Both  $\eta_1$  and  $\eta_2$  are related to the slenderness ratio and the eccentricity of the axial load and are defined as

$$\eta_1 = \eta_{10} + \left( 1 - 10 \frac{e}{D} \right) \geq 0.0 \quad (20)$$

$$\eta_2 = \eta_{20} + (1 - \eta_{20}) 10 \frac{e}{D} \leq 1.0 \quad (21)$$

where

$$\eta_{10} = 4.9 - 18.5\lambda + 17\lambda^2 \geq 0.0 \quad (22)$$

$$\eta_{20} = 0.25(3 + 2\lambda) \leq 1.0 \quad (23)$$

$$e = \text{eccentricity of axial loading} = \frac{M_{sd}}{P_{sd}} \quad (24)$$

where  $M_{sd}$  is the maximum design bending moment calculated by first order theory and  $P_{sd}$  is the design axial load. For concentric axial loading, eccentricity is zero. The slenderness parameters of the column is defined by

$$\lambda = \sqrt{\frac{A_s F_y + 0.85 A_c f_c' + A_r F_{yr}}{P_E}} \leq 2.0 \quad (25)$$

$P_E$  is the Euler buckling load defined as

$$P_E = \frac{(EI)_e \pi^2}{(kl)^2} \quad (26)$$

where,  $kl$  is buckling length of the column (effective length), and  $(EI)_e$  is effective bending stiffness

$$(EI)_e = E_s I_s + E_c I_c + E_r I_r \quad (27)$$

and  $E_s$  = modulus of elasticity of steel, MPa,  $E_c$  = modulus of elasticity of concrete, MPa,  $E_r$  = modulus of elasticity of reinforcing steel, MPa,  $I_s$  = moment of inertia of steel, mm<sup>4</sup>,  $I_c$  = moment of inertia of concrete (assumed to be uncracked), mm<sup>4</sup>,  $I_r$  = moment of inertia of reinforcing steel, mm<sup>4</sup>.

The modulus of elasticity for concrete defined as  $E_c = 0.8E_{cm}/\gamma_c$  where  $E_{cm}$  is the secant modulus of concrete, and  $\gamma_c$  is taken as 1.35.

The plastic resistance of composite cross-section,  $P_{pl}$ , which is reduced by  $\kappa$ , the buckling reduction factor, must be greater than the design load,  $P_{sd}$ .

$$P_{sd} \leq \kappa P_{pl} \quad (28)$$

where,  $\kappa$  = reduction factor accounting for the column slenderness ratio,  $P_{sd}$  = design value of the axial force, and  $P_{pl}$  = plastic resistance of the cross-section.

The buckling reduction factor  $\kappa$  is given in function of  $\lambda$ ,

$$\kappa = f_k - \sqrt{f_k^2 - \frac{1}{\lambda^2}} \leq 1.0 \quad (29)$$

where

$$f_k = \frac{1 - \alpha(\lambda - 0.2) + \lambda^2}{2\lambda^2} \quad (30)$$

where  $\alpha$  is imperfection factor equals 0.21 for concrete-filled circular and rectangular hollow sections.

#### 4.4 AIJ Code Method

This section introduces design formulas for CFT members shown in the 2001 edition of the AIJ standard for composite concrete and steel (SRC) structures [17, 18]. General descriptions are as follows:

- (1) The design method used in this standard is basically the allowable stress design supported by the elastic analysis of the structures. In earthquake-resistant design, it must be proved that the ultimate lateral load-resisting capacity of the allowable stress designed buildings is larger than the required value to resist a severe earthquake. The design loads and the allowable stresses of materials are specified by the Building Standard Law and AIJ standards [17].

- (2) The specified yield stress of steel tubes ranges from 235 MPa (215 if plate thickness  $t > 40\text{mm}$ ) to 355MPa (335 if  $t > 40\text{mm}$ ) in accordance with several steel grades which contain high-strength steel and centrifugal high-strength cast steel tube<sup>[17]</sup>.
- (3) The limiting value of the diameter-to-thickness ratio for a circular tube is as follows:

$$\frac{D}{t} \leq 1.5 \frac{23500}{F_y} \quad (31)$$

where  $D$  = depth or diameter of a circular tube,  $t$  = wall thickness of steel tube,  $F_y$  = standard strength to determine allowable stresses of steel and is the smaller of yield stress and 0.7 times tensile strength (MPa).

These values are relaxed to 1.5 times those of bare steels based on the research of the restraining effect of filling concrete on local buckling of steel tubes.

- (4) The long-term allowable bond stress between the filling concrete and the inside of the steel tube is 0.15 MPa for a circular tube and 0.1 MPa for a rectangular tube. The bond stress does not depend on the strength of the concrete. The values for the short-term stress condition are 1.5 times those for the long-term condition.
- (5) The allowable compressive stress of concrete  $f_{cc}$  is equal to  $F_c / 3$  for the long-term stress condition, and  $2F_c / 3$  for the short-term one, where  $F_c$  is the design standard compressive strength of concrete in MPa.
- (6) The maximum effective length  $kl$  of a CFT compression member is limited to:

$$\frac{kl}{D} \leq 50 \quad (32)$$

Allowable compressive strength of a CFT column is calculated by Eqs. 33 through 35.

$$\text{For } \frac{kl}{D} \leq 4; \quad N_{c1} = N_{cc} + (1 + \eta)N_{cs} \quad (33)$$

$$\text{For } 4 < \frac{kl}{D} \leq 12; \quad N_{c2} = N_{c1} - 0.125 \left\{ N_{c1} - N_{c3} \left( \frac{kl}{D} = 12 \right) \right\} \cdot \left( \frac{kl}{D} - 4 \right) \quad (34)$$

$$\text{For } \frac{kl}{D} > 12; \quad N_{c3} = N_{cc} + N_{cs} \quad (35)$$

where  $kl$  = effective length of a CFT column,  $D$  = width or diameter of a steel tube section,  $\eta = 0$  for a square CFT column,  $\eta = 0.27$  for a circular CFT column,  $N_{c1}$ ,  $N_{c2}$  and  $N_{c3}$  = allowable strengths of a CFT column,  $N_{cc}$  = allowable strength of a concrete column, and  $N_{cs}$  = allowable strength of a steel tube column .

$N_{c1}$  in Eq. 33 gives the cross-sectional allowable strength of a CFT column, in which the strength of the confined concrete is considered for a circular CFT column.  $N_{c3}$  in Eq. 35 gives the allowable buckling strength of a long column as the sum of the allowable buckling strengths separately computed for the filled-concrete and steel tube long columns.

Allowable compressive strength  $N_{cc}$  of a concrete column is calculated by Eqs. 36 and 37.

$$\frac{kl}{D} \leq 4; \quad N_{cc} = A_c \cdot f_{cc} = A_c \cdot \frac{F_c}{v_c} \quad (36)$$

$$\frac{kl}{D} > 12; \quad N_{cc} = \frac{N_{\sigma,cr}}{v_c} = \frac{A_c \cdot \sigma_{c,cr}}{v_c} \quad (37)$$

where  $A_c$  = cross-sectional area of a concrete column,  $f_{cc}$  = allowable compressive stress of concrete ( $= F_c / v_c$ ),  $F_c$  = design standard strength of filled concrete,  $v_c$  = factor of safety for concrete (3.0 and 1.5, for the long-term and short-term stress conditions respectively), and  $\sigma_{c,cr}$  = critical stress of a concrete column (see Eqs. 49 and 50).

Allowable compressive strength  $N_{cs}$  is calculated by Eqs. 38 through 40.

$$\text{For } \frac{kl}{D} \leq 4; \quad N_{cs} = A_s \cdot f_{cs} = A_s \cdot \frac{F_y}{v_s} \quad (38)$$

$$\text{For } \frac{kl}{D} > 12;$$

$$\lambda_s \leq \Lambda; \quad N_{cs} = A_s \cdot f_{cs} = \frac{A_s \left\{ 1 - 0.4 \left( \frac{\lambda_s}{\Lambda} \right)^2 \right\} F_y}{v_s} \quad (39)$$

$$\lambda_s > \Lambda; \quad N_{cs} = A_s \cdot f_{cs} = A_s \cdot \frac{0.6 F_y}{\left( \frac{\lambda_s}{\Lambda} \right)^2 v_s} \quad (40)$$

where  $A_s$  = cross-sectional area of a steel tube column,  $f_{cs}$  = allowable compressive stress of steel tube,  $\lambda_s$  = effective slenderness ratio of a steel tube,  $\Lambda$  = critical slenderness ratio =  $\pi \sqrt{E_s / 0.6 F_y}$ ,  $E_s$  = modulus of elasticity of steel,  $F_y$  = design standard strength of steel tube, and  $v_s$  = factor of safety for steel tube (long-term stress condition) which is given as below.

$$\text{For } \frac{kl}{D} \leq 4; \quad v_s = 1.5 \quad (41)$$

$$\text{For } \frac{kl}{D} > 12;$$

$$\lambda_s \leq \Lambda; \quad v_s = \frac{3}{2} + \frac{2}{3} \left( \frac{\lambda_s}{\Lambda} \right)^2 \quad (42)$$

$$\lambda_s > \Lambda; \quad v_s = \frac{13}{6} \quad (43)$$

For the short-term stress condition, 1.5 times the value for the long-term stress condition is used.

Ultimate compressive strength of a CFT column is calculated by Eqs. 44 through 46.

$$\text{For } \frac{kl}{D} \leq 4; \quad N_{cu1} = N_{\sigma, cu} + (1 + \eta)N_{s, cu} \quad (44)$$

$$\text{For } 4 < \frac{kl}{D} \leq 12; \quad N_{cu2} = N_{cu1} - 0.125 \left\{ N_{cu1} - N_{cu3} \left( \frac{l_k}{D} = 12 \right) \right\} \cdot \left( \frac{l_k}{D} - 4 \right) \quad (45)$$

$$\text{For } \frac{kl}{D} > 12; \quad N_{cu3} = N_{\sigma, cr} + N_{s, cr} \quad (46)$$

where  $kl$  = effective length of a CFT column,  $D$  = width or diameter of a steel tube section,  $\eta = 0$  for a square CFT column,  $\eta = 0.27$  for a circular CFT column,  $N_{cu1}$ ,  $N_{cu2}$  and  $N_{cu3}$  = ultimate strengths of a CFT column,  $N_{c, cu}$  = ultimate strength of a concrete column,  $N_{s, cu}$  = ultimate strength of a steel tube column,  $N_{c, cr}$  = buckling strength of a concrete column, and  $N_{s, cr}$  = buckling strength of a steel tube column.

$N_{cu1}$  in Eq. 44 gives the cross-sectional strength of a CFT column, in which the strength of confined concrete is considered for a circular CFT column.  $N_{cu3}$  in Eq. 46 gives the buckling strength of a long column as the sum of the buckling strengths separately computed for the filled-concrete and steel tube long columns.

Ultimate compressive strength  $N_{c, cu}$  and buckling strength  $N_{c, cr}$  of a concrete column are calculated by Eqs. 47 and 48, respectively.

$$N_{\sigma, cu} = A_c \cdot r_{cu} \cdot F_c \quad (47)$$

$$N_{\sigma, cr} = A_c \cdot \sigma_{\sigma, cr} \quad (48)$$

where  $A_c$  = cross-sectional area of a concrete column,  $F_c$  = design standard strength of filled concrete,  $\sigma_{\sigma, cr}$  = critical stress of a concrete column, and  $r_{cu} = 0.85$  = reduction factor for concrete strength. Critical stress  $\sigma_{\sigma, cr}$  is given by Eqs. 49 through 50.

$$\text{For } \lambda_{c1} \leq 1.0; \quad \sigma_{\sigma, cr} = \frac{2}{1 + \sqrt{\lambda_{c1}^2 + 1}} r_{cu} \cdot F_c \quad (49)$$

$$\lambda_{c1} > 1.0; \quad \sigma_{\sigma, cr} = 0.83 \exp\{C_c(1 - \lambda_{c1})\} r_{cu} \cdot F_c \quad (50)$$

where

$$\lambda_{c1} = \frac{\lambda_c}{\pi} \sqrt{\varepsilon_{cu}} \quad (51)$$

$$\varepsilon_{cu} = 0.93 (r_{cu} \cdot F_c)^{0.25} \times 10^{-3} \quad (52)$$

$$C_c = 0.568 + 0.00612 F_c \quad (53)$$

$\lambda_c$  = slenderness ratio of a concrete column

The ultimate compressive strength of a steel tube column is calculated by Eq. 54.

$$N_{s,cu} = A_s \cdot F_y \quad (54)$$

where  $A_s$  = cross-sectional area of a steel tube column, and  $F_y$  = design standard strength of steel tube.

Buckling strength of a steel tube column  $N_{s,cr}$  is calculated by Eqs. 55 through 57.

$$\text{For } \lambda_{s1} < 0.3; \quad N_{s,cr} = A_s \cdot F_y \quad (55)$$

$$0.3 \leq \lambda_{s1} < 1.3; \quad N_{s,cr} = \{1 - 0.545(\lambda_{s1} - 0.3)\} A_s \cdot F_y \quad (56)$$

$$\lambda_{s1} \geq 1.3; \quad N_{s,cr} = \frac{N_{s,E}}{1.3} \quad (57)$$

where

$$\lambda_{s1} = \frac{\lambda_s}{\pi} \sqrt{\frac{F_y}{E_s}} \quad (58)$$

$$E_{s,E} = \frac{\pi^2 E_s I_s}{k l^2} \quad (59)$$

$\lambda_s$  = slenderness ratio of a steel tube column,  $E_s$  = Young's modulus of steel tube, and  $I_s$  = cross-sectional moment of inertia of a steel tube column

## 5. Comparative study :

Using the above formulas, the nominal strengths of the columns are determined and denoted as  $P_c$ . The ratios of axial capacities obtained from the column tests ( $P_e$ ) to the predictions using the different methods ( $P_c$ ) as mentioned in the previous sections for the six experimental tests are given in **Table 3** and depicted in **Figure( 9)**.

Both **Fig. 9** and **Table 3** reveal the following:

- Generally, the predictions by the corresponding design methods have almost the same trend as they are greater than experimental results.
- AISC 1999 <sup>[19]</sup> code has a limited and constant allowance for concrete confinement, and become overly conservative for short columns (up to 41%). For large slenderness ratios, concrete confinement is minimal, column behavior is mainly elastic, and resistance models of the four design codes become less conservative.
- Although the beneficial confining effect has been taken into consideration in the design code of AISC 2010 <sup>[20]</sup>, still provides conservative results. The sectional capacity is about 27-33% lower than the experimental results.

- The AISC (1999) <sup>[19]</sup> was the most conservative, while Eurocode 4 <sup>[4]</sup> presented the values closest to the experimental results with a prediction range from 0-13% lower than the experimental results.
- AIJ <sup>[18]</sup> predicts a sectional capacity about 20-27% lower than the measured ultimate strengths. More conservative predictions are increasing with increasing slenderness ratios.
- Although there is significant effect of slenderness on the predicted ultimate axial strength of circular CFT columns using Eurocode 4 provisions, but it still giving the closest predications to the experimental results. The slenderness effect is very small using AISC (1999, 2010) <sup>[19, 20]</sup>. AIJ provisions are affected by the slenderness ratio in the same manner as those of Eurocode 4 provisions.
- It was found that the tested CFT columns under compression generally exhibited in a ductile manner, and the longitudinal force carried by the steel tube increased with the increase of the top endplate rigidity.

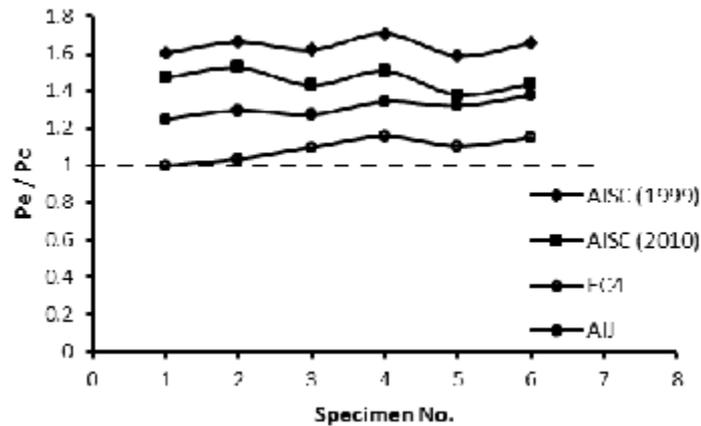
## 6. Summary and Conclusions :

From the investigation of the present study, the following conclusions can be drawn:

- The tested CFT columns under compression generally exhibited in a ductile manner, and the longitudinal force carried by the steel tube increased with the increase of the top endplate rigidity.

**Table 3 Comparison of experimental and code results**

No.	$P_e$ (kN)	$P_{c1}$ AISC 1999 (kN)	$P_{c2}$ AISC 2010 (kN)	$P_{c3}$ EC4 (kN)	$P_{c4}$ AIJ (kN)	$P_e /$ $P_{c1}$	$P_e /$ $P_{c2}$	$P_e /$ $P_{c3}$	$P_e /$ $P_{c4}$
1	1370	854.1	931.4	1370.3	1095.6	1.604	1.471	1.000	1.250
2	1420	854.1	931.4	1370.3	1095.6	1.663	1.525	1.036	1.296
3	1300	801.9	909.1	1184.1	1020.4	1.621	1.430	1.098	1.274
4	1370	801.9	909.1	1184.1	1020.4	1.708	1.507	1.157	1.343
5	1210	760.8	876.8	1095.1	915.8	1.590	1.380	1.105	1.321
6	1260	760.8	876.8	1095.1	915.8	1.656	1.437	1.151	1.376



**Fig.( 9) Comparison of experimental and code results**

- The deformation of the top part of steel tube becomes more obvious for the specimens with thicker endplate. That might occur due to stress concentrations at the contact surface.
- The shortest columns have attained their section capacities, whilst those slenderest columns only attained 85.7% of their section capacities. It is thus expected that slenderness reduction factors should be applied in designing slender CFT columns.
- The columns of  $L/D = 9.375$  reached an axial strain of half the value for columns of  $L/D = 2.5$ . Also, specimens with  $L/D = 6.25$  and  $9.375$ , presented global instability and the failure occurred at axial strain lower than specimens with lower  $L/D$  ratio.
- The lateral strain measured was about 1.5% for the columns with  $L/D = 9.375$  and 5% for the short columns at the ultimate load. Which indicates that specimens with  $L/D = 9.375$  exhibited insufficient lateral strain for mobilizing the confinement effect.
- The calculation of CFT columns' capacity revealed that the AISC (1999) is the most conservative and Eurocode 4 is the closest to the experimental results, which agree with what previous researches concluded [2]. However, the AISC code is simpler in calculations than both EC4 and AIJ, which may attract the designers to adopt.

In view of the foregoing, it is favorable to use Eurocode 4 analytical formulae to predict the axial capacity of circular CFT columns made with normal strength concrete and steel tubes. However, the computation is relative complex. AISC formulae are the simplest. It is recommended to make researches to find design formulas for composite columns that both accurate and simple.

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