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Experimental study on circular CFST short columns with intermittently welded stiffeners

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Abstract. This paper deals with the experimental study on strength the strength and deformation characteristics of short circular Concrete Filled Steel Tube (CFST) columns. Effect of vertical stiffeners on the behavior of the column is studied under axial compressive loading. Intermittently welded vertical stiffeners are used to strengthen the tubes. Stiffeners are attached to the inner surface of tube by welding through pre drilled holes on the tube. The variable of the study is the spacing of the weld between stiffeners and circular tube. A total of 5 specimens with different weld spacing (60 mm, 75 mm, 100 mm, 150 mm and 350 mm) were prepared and tested. Short CFST columns of height 350 mm, outer tube diameter of 165 mm and thickness of 4.5 mm were used in the study. Concrete of cube compressive strength 41 N/mm^2 and steel tubes with yield strength 310 N/mm^2 are adopted. The test results indicate that the strength and deformation of the circular CFST column is found to be significantly influenced by the weld spacing. The ultimate axial load carrying capacity was found to increase by 11% when the spacing of weld is reduced from 350 mm to 60 mm. The vertical stiffeners are found to effective in enhancing the initial stiffness and ductility of CFST columns. The prediction models were developed for strength and deformation of CFST columns. The prediction is found to be in good agreement with the corresponding test data.

Keywords: concrete filled steel tube column; circular CFST; welded stiffeners; vertical stiffeners; confinement

1. Introduction

Recently, Concrete Filled steel Tube (CFST) columns are widely used in construction due to their numerous advantages. Major advantage of using CFST column is its increased load carrying capacity compared to Reinforced Cement Concrete or Steel columns. Here, the concrete utilizes its confined compressive strength provided by the outer steel tube. Similarly, the core concrete delays the yielding of the thin steel tube by providing restrained from inside and there by arresting the inward buckling of tube. This will allow the steel to utilize failure stress higher than its yield strength. At ultimate load, the bond between steel tube and concrete fails and the tube starts to buckle outward and is reported in Schneider (1998), Han *et al*. (2001), Uy (2001), Han (2002) and Zhong (2006). In order to prevent this pre-mature failure, stiffeners are to be used. Various types of stiffeners are discussed below and Fig. 1 represents various such types.

Wang (2004), Xiao *et al*. (2005), Tao *et al*. (2007b), Park *et al*. (2011), Hu *et al*. (2011), Park and Choi (2013), Dong and Ho (2012), Ho and Lai (2013), Prabhu and Sundarraja (2013), Wang and Shao (2013), Lai and Ho (2014), Prabhu *et al*. (2015), Lai and Ho (2015a, b), Wang and Shao (2015), Li *et al*. (2016), Patel *et al*. (2016), Sulong and Shafigh (2016), Zhang *et al*. (2016) studied the influence of horizontal stiffeners such as rings, spirals, steel strips, CFRP/FRP sheets, vertical tubes and Tie bars on the load carrying capacity, deformation characteristics, ductility and stiffness of the CFST members. Based on the test results, it is found that the rate of strength degradation of CFST columns in post peak region can be effectively controlled using horizontal stiffeners. The studies by Tao *et al*. (2008, 2009), Dabaon *et al*. (2009), Petrus *et al*. (2010), Wang *et al*. (2017) and Zhu *et al*. (2017) used vertical stiffeners such as tab stiffeners and steel plates for improving the load and deformation characteristics of the concrete filled steel tube columns. It was observed that the vertical stiffeners enhance both bond strength and axial load capacity of concrete filled steel tube stub columns. Zhu *et al*. (2017) conducted experimental study on the stiffened square and rectangular CFST cold formed columns. In this work, sections were built up by welding four steel plates together to form square or rectangular tubes.

The studies by Tao *et al*. (2007a, 2008), Petrus *et al*. (2010), Wang *et al*. (2017), Zhu *et al*. (2017) showed that the ductility and axial strength can be enhanced considerably by providing continuously welded vertical stiffeners. The provision of continuous welding is assumed to be mandatory for achieving the design strength in the CFST columns. The vertical stiffeners reduce the chance of local buckling of tube in circular CFST short columns. However, the necessity of providing continuous or intermittent welding is not established explicitly. Considering this fact, intermittently welded stiffeners are provided in the present study. The effect of spacing of 4mm sized plug weld to connect the vertical stiffeners with the

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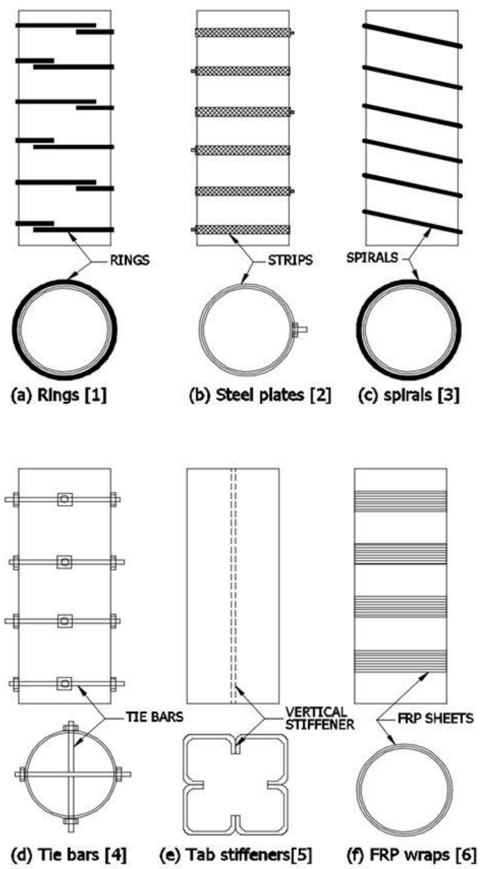

Fig. 1 Various methods of stiffening of short columns

outer steel tube on the deformation behavior of CFST column is studied. In circular tubes, plug weld is simple and easy way to connect the stiffeners in the interior zones away from the ends. The welding can be done from outside the tube through a pre-drilled hole of size 4mm on the steel tube and is shown in Fig. 2. This method of welding facilitates an easy and feasible method of construction, which can be adapted even for long and small diameter columns. An attempt is made in this study to predict the axial strength and deformation of short circular CFST columns with intermittently welded stiffeners.

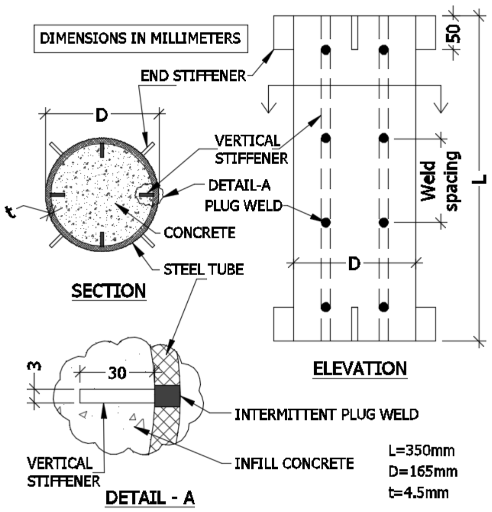

Fig. 2 Plan and section of the specimen

2. Materials

Concrete is filled in the steel tube for the preparation of CFST column. The concrete of compressive strength 41 N/mm^2 was prepared using the mix proportion of 0.45:1:1.78:2.06 (water : cement : fine aggregate : coarse aggregate). Ordinary Portland Cement (OPC) of 43 grade was used. River sand was used as fine aggregate and crushed granite stone of 12mm down size was used as coarse aggregate. Three standard cubes of size 150 mm × 150 mm × 150 mm were casted and tested on the same day of testing of CFST specimens to obtain the concrete compressive strength. A strip of steel tube was tested and found that the yield strength is 310 N/mm^2 and ultimate strength is 460 N/mm^2. Modulus of elasticity of steel sheet was determined by laboratory testing and was found to be 1.99×10^5 N/mm^2.

3. Details of specimen

Five CFST specimens were prepared in this study. The geometrical details of specimens are given in Fig. 2. Outer diameter (D) and tube wall thickness (t) of all steel tubes were 165 mm and 4.5 mm respectively. Four vertical stiffeners were provided in the CFST stub column specimens. The steel plates having a size of 350 mm × 30 mm × 3 m was used as vertical stiffeners. To avoid local bulging causing the elephant foot buckling failure,

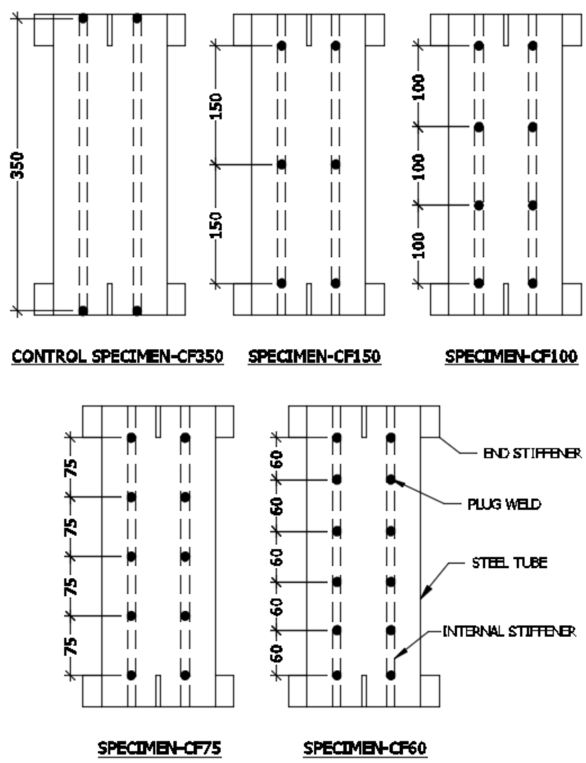

Fig. 3 Specimens with varying spacing of plug welds

(a) Steel tube with welded stiffeners (b) Finished top surface after concreting

Fig. 4 Specimen preparation

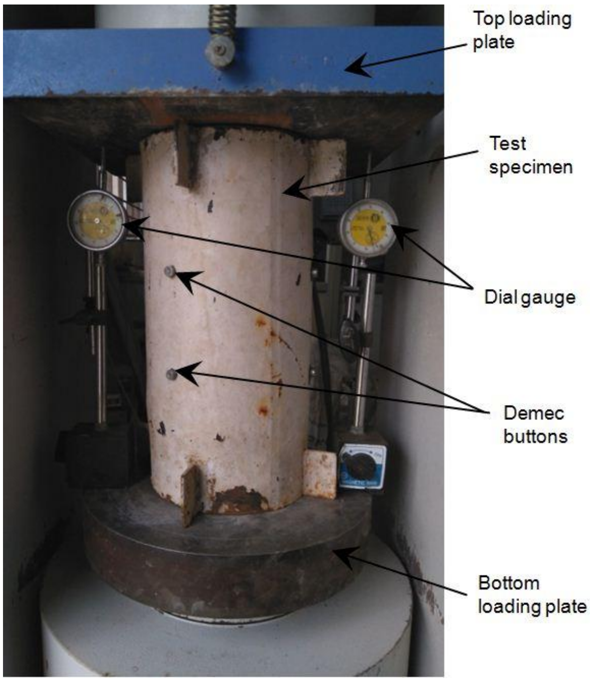

Fig. 5 Test setup of CF100

additional end stiffeners were provided outside the tube. The details of fixing end stiffeners and longitudinal vertical stiffeners are shown in Fig. 2. The spacing of 4 mm plug weld was varied between 60 mm to 350 mm. In controlspecimen, the vertical stiffeners was placed by welding it to the wall only at the ends. This is to maintain the area of steel constant in all specimens. The spacing of weld in specimens is shown in Fig. 3. The specimens are designated by the spacing of the weld in CFST tube with prefix 'CF'. For example, CF150 indicates that the CFST specimen with a weld spacing of 150 mm. The control specimen is designated as 'CF350'. Fig. 4 shows various stages of specimen preparation and concreting. A needle vibrator was used during concreting to ensure proper compaction inside the tube.

4. Testing

All specimens were cured for 28 days using moist burlap on the exposed concrete at top of the specimens. Specimens were cleaned after curing and tested using a 3000 kN capacity compression testing machine (Fig. 5). Axial deformations were measured with the help of a dial gauge of least count 0.002 mm. Verticality of the specimens were confirmed by ensuring same strains using demec gauge at four locations placed symmetrically all around the specimen. The axial load was applied concentrically. The load was applied in stages of 100 kN and vertical deformation of the specimen was recorded at every load stages. The specimens were loaded to fail. The failure load and the corresponding ultimate deformation were also recorded.

5. Results and discussion

A total of five specimens were tested and the load-deformation response was recorded. The failure load of the test specimens are given in Table 1.

The maximum increase was found to be for CF60 and the magnitude is 11% when compared to the control specimen (CF350). Hence, it is concluded that the welding of vertical stiffeners to the tube shell of the in-filled concrete column significantly influences its load carrying capacity. The variation of load carrying capacity due to the change in weld spacing is given in Fig. 6. The ratio of length to weld spacing is also shown in Fig. 6. The load carrying capacity is found to decrease with the increase in spacing. The strength of the short CFST column is inversely proportional to the spacing of the weld in the stiffener. It is

Table 1 Strength, stiffness and ductility of specimens

Specimen	N_{exp} (kN)	% increase in N_{exp}	K_i (kN/mm)	DI
CF350*	1870	-	525	3.01
CF150	1920	3	547	3.06
CF100	2020	8	584	3.33
CF75	2060	10	714	3.91
CF60	2080	11	728	4.09

*Control specimen

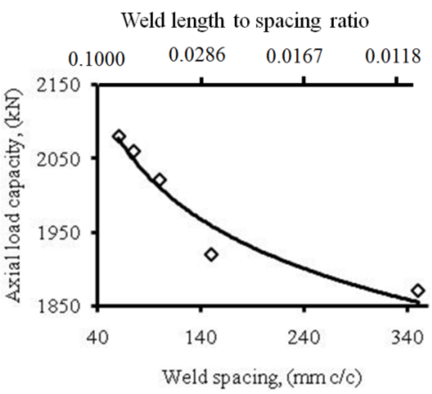

Fig. 6 Variation of axial load with weld spacing

expected that the strength will decrease significantly, if the spacing of the weld on stiffener is greater. This may be attributed to the fact that effective stress transfer between stiffener and steel shell taken place through the weld. The increase in the axial capacity is significant (about 10%) when the spacing is reduced from 350 mm to 75 mm. Thereafter, the increase was found to be small. This may be due to the fact that the stiffening required for the tube might have attained with the weld spacing of 75 mm. The welding creates synergic effects, which means, the welding of stiffener prevents the deformation of steel tube and the restrained steel tube provides more confinement to the concrete inside. The confinement of concrete is more effective when the stiffener deforms less, which is made possible by providing less spacing of weld. Greater confinement offer more load sustenance and the axial capacity is high when spacing decreases. It can be observed from the Fig. 6 that variation of axial load capacity of CFST column is non linear with the change in weld spacing. It is reported by (Chen and Zhang 2012, Tao *et al*. 2016, Zhang *et al*. 2012) the debonding between tube and concrete weakens the strength characteristics. Stiffeners helps to reduce the debonding separation compared to conventional CFST columns.

The load-deformation response of the test specimens is given in Fig. 7. It is observed that the control specimen (CF350) deforms more when compared to the other specimens. With the decrease in the spacing of welding to 60mm, the deformation is found to be the lowest. This indicates that the weld offer more stiffness to the composite column and axial stiffness of the composite column is found to increase with the decrease in spacing of weld. The weld prevented the local deformation of the sheet, which helped to offer more confinement to the concrete. Hence, it can be concluded that deformation of CFST columns can be controlled by proving appropriate spacing for the weld between tube and stiffeners. The reduction in the deformation corresponding to a load of 1500 kN in CF60

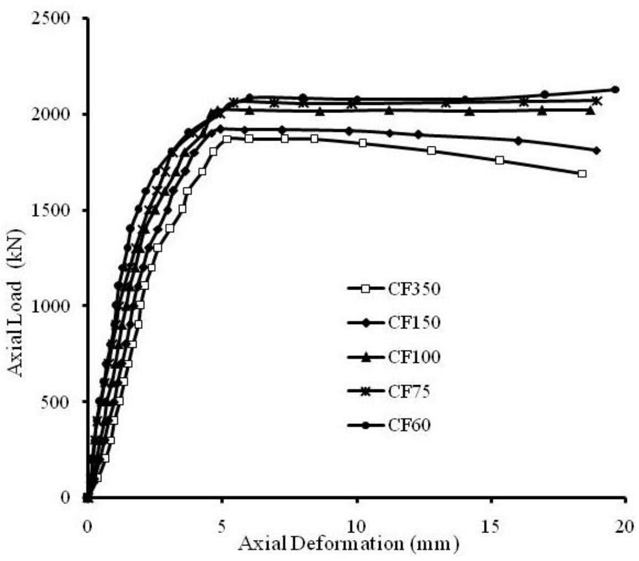

Fig. 7 Load – deformation response of in-filled tube column specimens

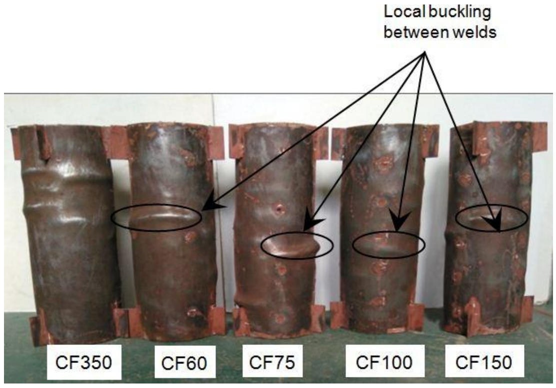

Fig. 8 Failure modes of tested specimens

specimen is found to be 18% when compared with the corresponding data of CF350.

The initial stiffness of CFST column, K_i can be calculated using the 1/3 tangent method applied on the load-deformation curves as explained by Park *et al*. (2011). The initial stiffness is given by

$$K_i = \frac{P_y}{\delta_y} \tag{1}$$

where P_y and δ_y are the axial load and corresponding deformation at yield point. The values calculated for the CFST specimens tested are listed in Table 1. It was observed that the axial stiffness of the columns increases with the reduction in weld spacing. The stiffeners participates in the load resistance during the initial stage itself. The weld in the stiffeners helps to redistribute the load and resist the overall deformation. This is the reason for increase in the stiffness with decrease in weld spacing. The increase in stiffness was found to be 38% when the spacing of weld in the CFST column is reduced to 60 mm from 350 mm. The initial stiffness of column having weld spacing of 75 mm and 60 mm is found to be almost equal. This may be due to the fact that the possibility of local deformation of the tube plate is fully arrested if the weld spacing is less than 75 mm.

The effect of stiffeners on the ductility of column is studied by calculating ductility index, DI as described by Park *et al*. (2013) is given in Eq. (2).

$$DI = \frac{\delta_{\max}}{\delta_y} \tag{2}$$

where $\delta_{\max}$ is the maximum deformation, calculated by 1/3 tangent method described in Park *et al*. (2013). Ductility was found to improve when the stiffeners are introduced in the CFST specimens. The ductility index of various test specimens are given in Table 1. The increase in the ductility was found to be 36% corresponding to the weld spacing of 60 mm when compared to control specimen. Failure modes of the specimens tested are shown in Fig. 8. It was observed that all specimens failed due to local buckling of the tube wall. The outward bulging were found between weld points in the stiffened columns and is shown in Fig. 8.

6. Prediction of ultimate load

Ultimate load of the CFST column is predicted using ACI and EC4 method. A model is proposed to predict the axial load of CFST column with welded stiffeners.

6.1 ACI 318 Model

The axial strength of a short composite compression member is calculated as

$$N_{ACI} = 0.85 A_c f_c' + A_a f_{yt} \tag{3}$$

where, A_c and A_a are the area of concrete and steel respectively, f_c' and f_{yt} are the cylinder compressive strength of concrete and yield strength of steel tube respectively.

6.2 Eurocode 4 (EC4) Model

Axial load capacity of concrete filled steel circular tube column is given by

$$N_{EC4} = \eta_a A_a f_{yt} + A_c f_c' \left(1 + \eta_c \frac{t}{d} \frac{f_{yt}}{f_c'}\right) \tag{4}$$

where, 't' is the tube thickness and 'd' is the tube diameter.

$$\eta_a = 0.25(3 + 2\bar{\lambda}) \tag{4a}$$

$$\eta_c = 4.9 - 18.5\bar{\lambda} + 17\bar{\lambda}^2 \tag{4b}$$

$$\bar{\lambda} = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}} \tag{4c}$$

$$\text{where} \quad N_{pl,Rk} = A_a f_{yt} + A_c f_{ck}' \tag{4d}$$

$$\text{and} \quad N_{cr} = \frac{\pi^2 (EI)_{eff}}{l^2} \tag{4e}$$

$$(EI)_{eff} = E_a I_a + K_e E_{cm} I_c \tag{4f}$$

E_a is the modulus of elasticity of steel tube and I_a is its second moment of area. K_e is 0.6, a factor, E_{cm} is the secant modulus of concrete and I_c is its un-cracked second moment of area, l is the effective length of column.

$$E_{cm} = 22 \left[\frac{f_{cm}}{10}\right]^{0.3} \tag{4g}$$

$$\text{where} \quad f_{cm} = f_c' + 8 \tag{4h}$$

It may be noted that the effect of the spacing of the weld is not accounted for in this model given by EC4.

6.3 Proposed model

To account for the synergic effect of the welding of stiffeners, a confinement modification factor (k_c) is proposed.

The Eq. (4) is modified as

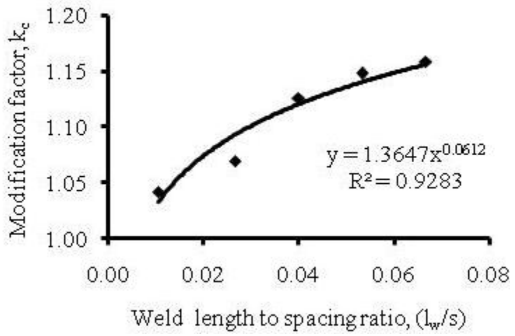

Fig. 9 Variation of modification factor with weld length to spacing ratio

$$N_p = S\, k_c \left(\eta_a A_a f_{yt} + A_c f_c' \left(1 + \eta_c \frac{t}{d} \frac{f_y}{f_c'}\right)\right) \tag{5}$$

where S is the shape factor of cross section of column, S is taken as 1.0 for circular cross section and 0.7 for the square cross section. The magnitude of S is determined based on the test data of Dabaon *et al*. (2009) and Zhu *et al*. (2017). The lower value of S for square section is attributed to the lower confinement mobilized in the tube when compared to the circular tube. The statistical regression analysis is carried out to determine model for k_c and is given in Fig. 9.

The k_c is given by

$$k_c = 1.3647 \left({}^{l_w}/_{S}\right)^{0.0612} \tag{6}$$

7. Prediction of axial deformation

The axial deformation of the column is predicted using the equation

$$\delta_{pi} = \frac{N_i l}{A_e E_i} \tag{7}$$

where N_i is the axial load on column, l is the length of the column, A_e is the equivalent cross sectional area of column.

A_e in Eq. (7) is given by

$$A_e = m A_a + A_c \tag{7a}$$

m is the modular ratio. In Eq. (7), E_i is the secant modulus of elasticity corresponding to the load step N_i. The secant modulus of elasticity of the concrete is determined based on the experimental data. The statistical multi-regression analysis was carried out to determine the prediction model for secant modulus and is given by

$$E_i = E_{cm} \left\{0.1972 - 0.1554 \left(\frac{N_i}{N_{pu}}\right)^2 + 0.1374 \left(\frac{N_i}{N_{pu}}\right) - 0.0022 \left(\frac{S}{l_w}\right)^{\frac{N_i}{N_{pu}}}\right\} \tag{8}$$

Table 2 Comparison of experimental results with predicted values

Reference	Sl. No.	Specimen	N_{exp}/N_{ACI}	N_{exp}/N_{EC4}	N_{exp}/N_p
Present study	1	CF350	1.40	1.04	1.00
	2	CF150	1.43	1.07	0.98
	3	CF100	1.51	1.13	1.00
	4	CF75	1.54	1.15	1.01
	5	CF60	1.55	1.16	1.00
Dabaon *et al*. (2009)	1	SHS1C30	1.26	1.17	1.06
	2	SHS1C60	1.17	1.06	1.01
	3	SHS2C30	1.37	1.25	1.27
	4	SHS2C60	1.15	1.03	1.52
Zhu *et al*. (2017)	1	Pb-6-1	1.09	1.05	0.83
	2	Pc-6-1	0.93	0.90	0.73
	3	Pd-6-1	1.30	1.27	1.06
	4	Pe-6-1	1.31	1.27	1.10
	5	Pb-6-2	1.20	1.16	0.90
	6	Pc-6-2	1.23	1.20	0.97
	7	Pd-6-2	1.29	1.26	1.06
	8	Pe-6-2	1.27	1.23	1.06
	9	Pb-6-3	1.12	1.08	0.84
	10	Pc-6-3	1.20	1.17	0.94
	11	Pd-6-3	1.24	1.20	1.00
	12	Pe-6-3	1.35	1.32	1.14
	13	Pb-10-1	1.26	1.23	0.95
	14	Pc-10-1	1.20	1.17	0.94
	15	Pd-10-1	1.10	1.08	0.90
	16	Pe-10-1	1.11	1.08	0.93
	17	Pb-10-2	1.22	1.19	0.92
	18	Pc-10-2	1.15	1.13	0.90
	19	Pd-10-2	1.15	1.13	0.94
	20	Pe-10-2	1.09	1.07	0.92
	21	Pb-10-3	1.25	1.22	0.94
	22	Pc-10-3	1.19	1.17	0.94
	23	Pd-10-3	1.15	1.13	0.94
	24	Pe-10-3	1.12	1.10	0.94
Average			1.24	1.15	0.99

where N_{pu} is the ultimate axial load predicted using Eq. (5). In Eqs. (6) and (8), s is the weld spacing and l_w is the length of weld.

8. Comparison of prediction with the experimental data

The predicted axial load capacity and deformation of CFST column is compared with the corresponding experimental results.

8.1 Axial capacity

A comparison is made between the experimental load (N_{exp}) and the predicted load by ACI code (N_{ACI}), Eurocode (N_{EC4}) and the proposed method (N_p). The comparison is given in Table 2. The ratio of the experimental to predicted strength using ACI method $\left(\frac{N_{exp}}{N_{ACI}}\right)$ is found to range between 0.93 and 1.55. Similarly, the ratio that with the predicted strength using EC4 $\left(\frac{N_{exp}}{N_{EC4}}\right)$ varies from 0.90 and 1.32. It varies from 0.73 and 1.52 in the case of the ratio of the experimental strength to predicted strength using proposed method $\left(\frac{N_{exp}}{N_p}\right)$. The average values of the ratios $\frac{N_{exp}}{N_{ACI}}$, $\frac{N_{exp}}{N_{EC4}}$ and $\frac{N_{exp}}{N_p}$ is found to be 1.24, 1.15 and 0.99 respectively. This indicates that the strength prediction using equation (Eq. (5)) is in good agreement with experimental results.

8.2 Axial deformation

The load -deformation response of the concrete in-filled steel tube short column is predicted. The predicted axial deformation is found to be in good agreement with the experimental data. The comparisons are shown in Fig. 10. The non-linear degradation of stiffness with the increase in load is being simulated using the proposed model. The deformation is found to be large at ultimate stage. The deformation of the concrete in-filled short column at ultimate stage is predicted by substituting $\left(\frac{N_i}{N_{up}}\right)$ is equal to 1 in Eqs. (7) and (8).

The magnitude of the predicted ultimate deformation is compared with the corresponding experimental data and is shown in Table 3. The ultimate axial deformation reported by Dabaon et al. (2009) is also predicted using the proposed model given by Eq. (7) and is given in Table 3. The magnitude of ratio of $\frac{\delta_{ue}}{\delta_{up}}$ for the columns SHS1C30 and SHS1C60 by Dabaon *et al*. (2009) was found to be 0.97 and 2.94 respectively. The variation may be due to the variation in confinement offered by square hollow section tube used in their study. The average value of the ratio of experimental data of axial deformation at ultimate state to that of predicted is found to be 1.40. This indicates that the prediction is comparable with the experimental data.

The load-deformation response of a CFST column having continuously welded stiffener (CF0) is also predicted using the proposed model and is given in Fig. 11. The non-linear deflection response is obtained. Hence, it can be concluded that the proposed model is useful for predicting the deflection response of CFST short columns with continuous weld.

9. Conclusions

Following conclusions were drawn from the experimental study conducted on circular CFST short columns with welded stiffeners.

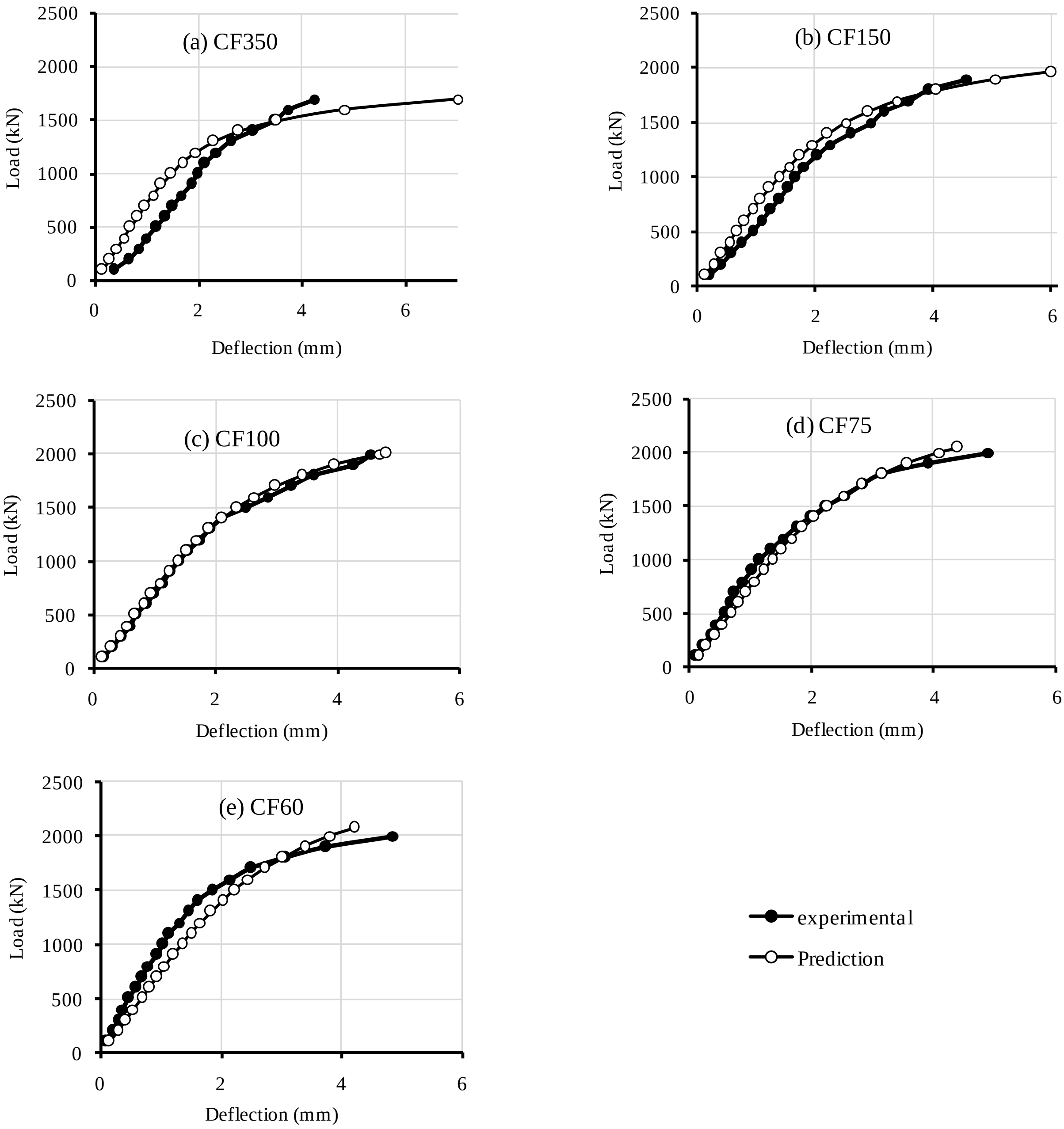

Fig. 10 Comparison of predicted deformations with the experimental data

Table 3 Comparison of experimental deformations with predictions

Reference	Sl. No.	Specimen designation	δ_{ue}/δ_{up}
Present study	1	CF350	1.74
	2	CF150	1.21
	3	CF100	1.06
	4	CF75	0.93
	5	CF60	0.91
Dabaon *et al*. (2009)	1	SHS1C30	0.97
	2	SHS1C60	2.94
Average			1.40

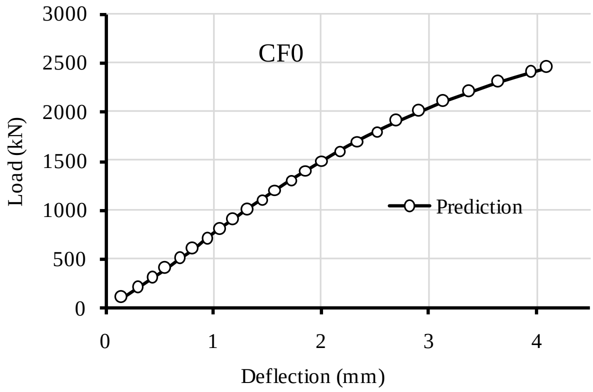

Fig. 11 Predicted deformation for specimen similar to test specimen having continuous weld

- The spacing of weld of stiffeners significantly influences the load carrying capacity of CFST columns.
- The axial stiffness and ductility of column is found to be increasing with decrease in weld spacing. The axial deformation of CFST column decreases with decrease in weld spacing.
- ACI and EC4 underestimate the axial load carrying capacity of CFST short columns.
- The prediction based on proposed model for axial load (Eq. (5)) is found to be in good agreement with the experimental data. The model accounts for the shape of the tube and the confinement of concrete.
- The load-deformation response of CFST short column is well predicted using the proposed model. The proposed model is capable of predicting the load-deformation response of short CFST column having continuously welded stiffeners.

It is expected that a designer can assess the load carrying capacity of the CFST short column accurately and perform economical design using the proposed model. The estimate on the deformation may help to carry out the design corresponding to serviceability limits of deflection of structural members. The optimization of stiffeners intermittently welded to tubes of CFST column may be studied in the future. The guidelines for the design of concrete filled steel tube columns are not yet included in the current Indian Standard code for plain and reinforced concrete (IS: 456, 2000). It is expected that the recommendations of the present study would help the committee of IS: 456 (2000) to bridge the gap. The accuracy of the regression model can be improved by accounting for the results of the parametric study of the finite element simulation based on the finite element simulation of the CFST columns.

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