Finite Element Analysis and Theoretical Investigations on Concrete Filled Steel Tubular Columns

S. Jayalekshmi[1] and Sankar Jegadesh J. S.[2]

[1]Associate Professor, [2]Research Scholar, Department of Civil Engineering,
National Institute of Technology, Tiruchirappalli, Tamil Nadu, India

Abstract-This paper presents the behavior and design of axially loaded concrete filled steel tubular (CFST) columns. A numerical investigation on the behavior of concrete filled steel tubular columns loaded in axial compression is presented. Nonlinear finite – element analysis is performed using commercial software ANSYS. The numerical models are used for the computations and the results are validated with the corresponding experimental program from the literature. It is observed that the numerical model is able to map the load deflection response of the CFST specimens. A good agreement is also observed between the experimental and the predicted numerical results. The column strength predicted from the finite element analysis and by using the Lu and Zhao (2008) are compared with the corresponding experimental results obtained from the literature. The comparative results ensured that D/t ratio plays a prominent role on the compressive behavior of the CFST specimens. This paper quantifies the difference between the experimental and numerical results, and the ultimate load of the CFST columns estimated by various International code procedures.

Keywords: CFST Columns, Numerical Model, Nonlinear Analysis, ANSYS, Load Deflection Response, Comparison of International Codes

I. Introduction

CFST columns are often subjected to combined axial load and an extensive research has been conducted on the behavior of CFST columns subjected to axial load. Tests of concrete-filled carbon steel tube columns were conducted by Schneider (1998), Uy (2004), Huang et al (2002). , Han and Yao (2003), Mursi and Uy (2003), Uy (2001) , Sakino et al.(2004), Giakoumelis and Lam(2004) and many other researchers. These tests were carried out on concrete-filled carbon mild steel and high strength steel tube columns using circular, square and rectangular hollow sections. The experimental study of axially loaded short CFST columns with depth-to-thickness ratio ranging from 17 to 50 was undertaken by Schneider (1998). Fourteen specimens were examined and the effects of steel tube shape and tube wall thickness on the ultimate axial strengths of CFST columns were analysed. Circular steel tubes possessed higher post-yield axial ductility and strength than square or rectangular concrete-filled tube sections. All circular CFST columns exhibited strain hardening. Strain hardening occurred in rectangular CFST columns with D/t < 20. Sakino et al. (2004) observed early concrete crushing and local buckling for normal strength concrete compared to high strength concrete. Georgios Giakoumelis and Dennis Lam (2004) identified that for high-strength CFT columns, the peak load was observed for small shortening whereas for normal concrete the ultimate load was obtained with large displacement. This paper primarily aims to present a numerical model for the analysis of the CFST columns. The model is validated by comparison with experimental results, from literature. The experimental data from the literature is then used to verify the accuracy of Lu and Zhao (2008) theoretical equation.

II. Numerical Modeling of CFST Columns

The contact between steel tube and concrete causes composite action between steel and concrete in a CFST column. The radial lateral confining pressure exerted by the steel tube on the concrete induces confinement in concrete. A biaxial state of stress is also induced in steel, which may cause some reduction in its axial load capacity. Thus any numerical model that intends to capture the behaviour of CFST in compression must use suitable constitutive model for the steel tube and concrete. Further, to effectively replicate the inherent advantages of CFST, it is necessary that the composite action between steel and concrete be very carefully modeled.

The uniaxial behavior of the steel is simulated by an elastic-perfectly plastic model. The equivalent uniaxial stress–strain curves for both unconfined and confined concrete are shown in Fig 1 , where f_c is the unconfined concrete cylinder compressive strength, which is equal to 0.8(f_{cu}), and f_{cu} is the unconfined concrete cube compressive strength. The corresponding unconfined strain (ε_c) is taken as 0.003. The confined concrete compressive strength (f_{cc}) and the corresponding confined strain (ε_{cc}) can be determined from the following equations respectively, (Mander et al., 1988) $f'_{cc} = f'_c + k_1 f_1$ and $\varepsilon'_{cc} = \varepsilon'_c (1 + k_2 \frac{f_1}{f'_c})$. The coefficients k_1 and k_2 are constants and can be obtained from experimental data. Meanwhile, the constants k_1 and k_2 were set as 4.1 and 20.5. where f_l represents the confining pressure around the concrete core calculated from the following empirical equations (Hu et al., 2003):

$$\frac{f_1}{f'_y} = [0.055048 - 0.001885\,(B/t)] \quad \text{for } 17 \leq B/t \leq 29.2 \quad \text{and} \quad \frac{f_1}{f'_y} \text{ for } 29.2 \leq \frac{B}{t} \leq 150 . \qquad (1)$$

To define the full equivalent uniaxial stress –strain curve for confined concrete as shown in Figure 1, three parts of the curve have to be identified. The first part is the initially assumed elastic range to the proportional limit stress. The proportional limit stress is taken (0.5 f_{cc}) (Hu et al., 2003). The initial Young's modulus of confined concrete (E_{cc}) is reasonably calculated using the empirical equation by ACI, 1999. The Poisson's ratio (υ_{cc}) of confined concrete is taken as 0.2.$E_{cc} = 4700\sqrt{f_{cc}}\ MPa$ The second part of the curve is the nonlinear portion starting from the proportional limit stress 0.5(f_{cc}') to the confined concrete strength (f_{cc}') .This part of the curve can be determined from (6), which is a common equation proposed by,

$$f = (E_{cc}\,\varepsilon) \,/\, \left\{1 + (R + R_E - 2)\left(\frac{\varepsilon_{c'}}{\varepsilon_{cc'}}\right) - (2R-1)\left(\frac{\varepsilon_c}{\varepsilon_{cc'}}\right)^2 + R\left(\frac{\varepsilon_c}{\varepsilon_{cc'}}\right)^3\right\} \quad \text{Where:} \quad R = \frac{R_E\,(R_\sigma - 1)}{(R_\varepsilon - 1)^2} - \frac{1}{R_\varepsilon} \quad \text{and}$$

$R_E = \frac{E_c\ \varepsilon_{cc'}}{f'_{cc}}$ While the constants are taken equal to 4 (Ell body et al. 2006), this part starts from 0.5(f'_{cc}) and progresses to the confined concrete strength (f'_{cc}). The last part of the curve, the descending line is assumed to be terminated at the point where $f_c = k_3 f'_{cc}$ and $\varepsilon_c = 11.\ \varepsilon_{cc}'$. k_3 can be calculated, from the following empirical equations (Hsuan et al., 2003):

$$k_3 = 1 \ for\left(21.4 \leq \frac{D}{t} < 40\right);\ k_3 = 0.0000339\left(\frac{D}{t}\right) - 0.010085\left(\frac{D}{t}\right) + 1.3491 \ \ for\ \left(40 \leq \frac{D}{t} \leq 150\right) \qquad (2)$$

Because the concrete in the CFT columns is usually subjected to triaxial compressive stresses, the failure of concrete is dominated by the compressive failure surface expanding with increasing hydrostatic pressure. Hence, a Drucker-Prager (DP) yield criterion is used to model the yield surface of concrete which assumes an elastic perfectly plastic response. The bilinear kinematic hardening model was used to simulate the stress–strain curve of steel and assumed to be an elastic-perfectly plastic material. The bilinear model requires the yield stress (f_y) and the hardening modulus of the steel (E_s), the Constitutive law for steel behavior is $\sigma_s = E_s\varepsilon_s,\ \varepsilon_s \leq \varepsilon_y$ and $\sigma_s = f_y + E_s{}'\varepsilon_s{}',\ \ \varepsilon_s > \varepsilon_y$ Where: σ_s is the steel stress, s is the steel strain, Es is the elastic modulus of steel, Es' is the tangent modulus of steel after yielding, $E_s' = 0.01Es$, f_y and ε_y are the yielding stress and strain of steel, respectively.

III. Finite Element Modeling - Ansys

Numerical simulations were carried out with finite element software ANSYS, where the concrete and steel were modeled as following: An eight-node solid element, Solid65, is used to model the concrete. SHELL181 is suitable for analyzing thin to moderately-thick shell structures. All the specimens were modeled as 3D structural elements. The two ends were assumed to be hinged, for modeling. At both the ends, displacement in x, y directions (U_x, U_y) were restrained and translation U_z as well as rotational degrees of freedom in x, y, z directions were considered to be free**.** The coefficient of friction between the concrete and steel surfaces was given by 0.25. Rigid Contact is provided between the two surfaces, that enables the pressure to be transmitted across the two surfaces, only when there is actual contact among them, while causing the surfaces to separate under the influence of a tensile force. Thus, the correct simulation of composite action between concrete and steel tube is achieved successfully in the prediction of the behavior of the CFST column.

Theoretical Investigations- Lu and Zhao (2008)

Lu and Zhao (2008) proposed the axial capacity of CFST columns based on AIJ code. The axial compressive stength of the CFST column is given by; $N_{cu} = A_c f_{cp}(1 + \eta_c) + A_s f_y$ where, η_c is the coefficient of confinement of the concrete, $\eta_c = 1.25 \frac{t}{D} (f_y/f_{cp})$ and f_{cp}= unconfined compressive strength of the concrete , $f_{cp} = \Upsilon_c f_c' = 1.67 D_c^{-0.112} f_c'$ where, Υ_c = strength reduction factor introduced to the scale effect into consideration; f_c'= the unconfined cylinder compressive strength of concrete; D_c= diameter of the core.

IV. Comparison of Results and Discussion

The estimated capacities of the CFST specimens using the numerical model and the previously described equations are compared to the experimental result from the literature as shown in Table 1.

Results and Validation of the Numerical Model

The numerical model was used to simulate the CFST columns. The accuracy of the numerical model was evaluated based on the difference between the experimental data and ANSYS peak load. The dimensions and other specifications of the CFST columns are shown in Table1. A comparison of peak load capacity of CFST specimens are presented in Table 1. Figure 2, 3 and 4 compare the experimental and ANSYS load-deflection pattern for circular, square and rectangular CFST columns respectively. The experimental and numerical peak load values also display good agreement with each other. The results show that the numerical model is capable of reproducing the load-deformation behavior. The load capacity P_{ANSYS} determined numerically is varying within ± 10% of the experimental values.

It is observed that the improvements in the axial capacity of CFST specimens are more affected by the change in the D/t ratio of the steel tube. From Table 1, it is observed that the CFST's axial capacity decreases as the D/t increases due to the reduction in the confinement provided by the smaller wall thickness. The average value and standard deviation of P_{exp}/P_{ANSYS} were obtained as 1.07 and 0.05 respectively, which indicates a good correlation between the experimental and numerically simulated results. Based on the above comparison, it can be concluded that the FE model is sufficiently verified and can be used to conduct further comprehensive studies to investigate the effect of additional parameters influencing the capacity of CFST columns.

Comparison of Results with Lu and Zhao (2008)

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 equation proposed by Lu and Zhao (2008). Fig 5-7 indicates that the values predicted by this method are in good agreement with the experimental results for circular CFT stub columns. The average value and standard deviation of $P_{exp} / N_{Lu\ and\ Zhao}$ were obtained as 1.04 and 0.06 respectively, which indicates a good correlation between the experimental and numerically simulated results.

 ISBN : 978-81-929742-0-0

Table 1. Comparison of Numerical and Theoretical Results with Experimental Results

Specimen Label	Diameter D (mm)	H(mm)	Thick-ness t, (mm)	Length L, (mm)	f_c (MPa)	f_y (MPa)	E_s (GPa)	Experimental Load(kN) (Schneider, 1998) P_{exp}	Numerical Load (kN) P_{FEM}	Theoretical Load (kN) $P_{Lu\ and\ Zhao}$
C1	140.8	-	3.00	605.4	28.180	285	189.475	881	752	748.2239
C2	141.4	-	6.50	608.0	23.805	313	206.011	1825	1763	1301.39
C3	140.0	-	6.68	616.0	28.180	537	205.322	2715	2638	2143.348
S1	127.3	127.3	3.15	611.0	30.454	356	180.518	917	863	1020.998
S2	126.9	126.9	4.34	609.0	26.044	357	190.164	1095	986	1220.219
S3	126.9	127.0	4.55	609.0	23.805	322	205.322	1113	1074	1141.770
S4	125.3	126.5	5.67	601.0	23.805	312	203.944	1202	1158	1286.100
S5	126.8	127.2	7.47	608.6.0	23.805	347	204.633	2069	1789	1759.425
R1	152.3	76.6	3.00	609.2	30.454	430	190.164	819	743	893.8728
R2	152.8	76.5	4.47	611.0	26.044	383	213.59	1006	932	1059.110
R3	152.4	101.8	4.32	609.6	26.044	413	214.968	1144	1092	1304.441
R4	152.7	102.8	4.57	610.0	23.805	365	206.01	1224	1177	1218.529
R5	151.4	101.3	5.72	606.0	23.805	324	204.63	1335	1287	1286.155
R6	152.37	102.13	7.34	609.0	23.805	358	205.32	1691	1634	1710.764

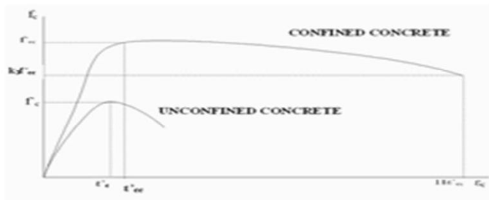

Fig. 1. Uniaxial Stress-Strain Curves for Confined and Unconfined Concrete

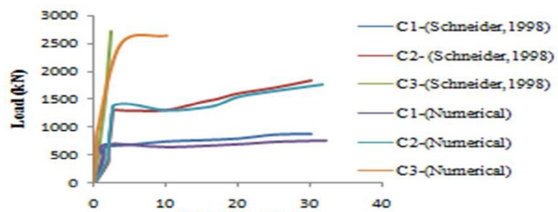

Fig.2 Load – Deflection Response : Circular CFST Specimen

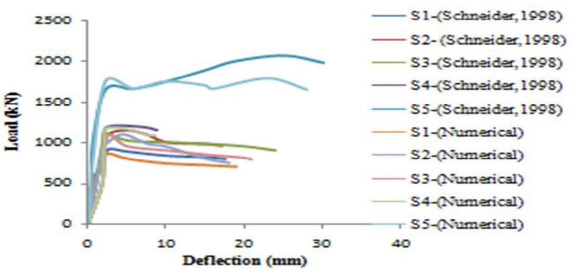

Fig.3 Load – Deflection Response : Square CFST Specimen

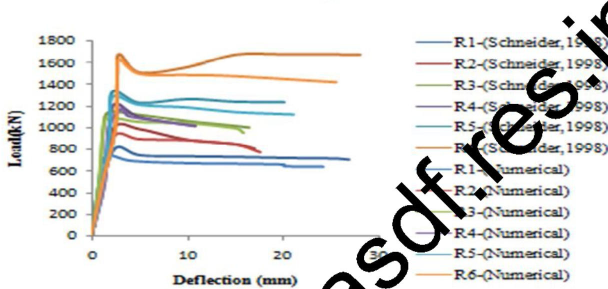

Fig.4 Load – Deflection Response : Rectangular CFST Specimen

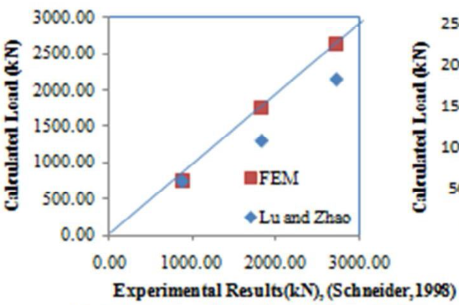

Fig.5 Experimental Results Vs Theoretical Results: Circular CFST Specimens

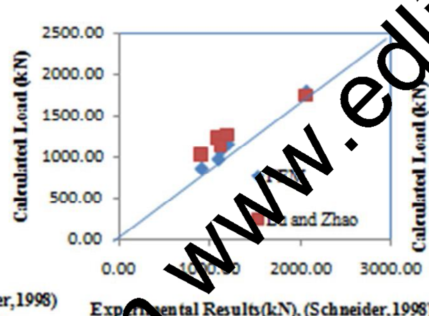

Fig.6 Experimental Results Vs Theoretical Results: Square CFST Specimens

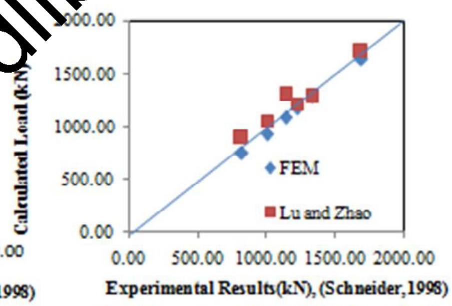

Fig.7 Experimental Results Vs Theoretical Results: RectangularCFST Specimens

V. Conclusions

A nonlinear FEM based model has been developed using ANSYS for the numerical simulation of CFST specimens. The numerical model is validated by comparing its output with the corresponding experimental data available in literature. Good agreement between the experimental data and the model's output in terms of the axial capacity is observed. The established FE model can be used for subsequent parametric studies that would shed more light on additional factors influencing the CFST behaviour. The axial strength of CFST columns are predicted by Lu and Zhao (2008) equation. A comparison of the experimental results with the numerical analysis and theoretical results is presented in this study. From the observations made, it is observed that the Lu and Zhao (2008) equation predicted the compression capacity of CFST columns conservatively and can be used to design due to their inherent conservatism. Thus, this paper provides a comprehensive summary of the various International code procedures and a comparative study used to estimate the capacity of CFST columns.

References

[1]. Giakoumelis, G., and Lam, D. (2004). Axial capacity of circular concrete-filled tube columns, Journal of Constructional Steel Research, 60:7, 1049-1068.

[2]. Han, L. H., Yao, G. F., and Zhao, X. L. (2005). Experiment behavior of thin-walled hollow structural steel (HSS) stub columns filled with self-consolidating concrete (SCC), Journal of Constructional Steel Research, 61:9, 1241-1269.

[3]. Hu, H. T., Huang, C. S., Wu, M. H., and Wu, Y. H. (2 003). Nonlinear analysis of axially loaded concrete-filled steel tube columns with confinement effect. Journal of Structural Engineering, 129:10, 1322-1329.

[4]. Huang, C. S., Yeh, Y. K., Hu, H. T., Tsai, K. C., Weng, Y. T., and Wang, S. H., and Wu, M. H. (2002). Axial load behavior of stiffened concrete-filled steel columns, Journal of Structural Engineering, 128:9, 1222-1230.

[5]. Lu Z.H. and Zhao Y.G., (2008). Mechanical behavior and ultimate strength of circular Cft columns subjected to axial compression loads.14th World Conference on Earthquake Engineering, Beiging, China.

[6]. Mander, J. B., Priestley, M. J. N., and Park, R. (1988). Theoretical stress-strain model for confined concrete, Journal of Structural Engineering, 114: 8, 1804-1826.

[7]. Sakino, K., Nakahara, H., Morino, S., and Nishiyama, I. (2004). Behavior of centrally loaded concrete-filled steel-tube short columns, Journal of Structural Engineering, 130:2, 180-188.

[8]. Schneider S. P. (1998). Axially lo aded concrete-filled steel tubes, Journal of Structural Engineering, 124:10, 1125-1138.

[9]. Uy B. (2001). “Strength of short concrete filled high strength steel box columns.” J. of Constructional Steel Research; 57(2):113–34.