Strength of short concrete-filled steel tubes considering a physic-geometrical factor of concrete core under axial compression

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Abstract. Test data have shown that the increase in strength of concrete-filled steel tubular (CFST) columns due to confinement are in variable bigger range than the strength without confinement. Different approaches and design philosophies were adopted in different design codes to account for this increase in strength. The theoretical research herein consider the effect of important physic-geometrical concrete core factors, such as the effect of confining lateral pressure on the concrete core expressed through the coefficient value of lateral confinement, the variable expected direction of failure planes and the enhanced compressive strength of concrete core with different concrete grades. Analytical expression has been proposed to determine the compressive strength of short (CFST) columns taking into account these factors for normal weight concrete core. The results of the analysis and comparison with some design codes indicate that the proposed approach yields satisfactory prediction.

Keywords: concrete-filled steel tubular columns; physic-geometrical factor; coefficient lateral confinement; and direction of failure planes

INTRODUCTION

The continuously expanding application of reinforced concrete in building accompanies by creating new structures of different forms and shapes, with special interest being directed towards advanced composite materials and systems. Concrete filled steel structures are one of these development trends and nowadays have been used in a diversity of applications, including piles, piers for bridges, and in earthquake-resistant structures. In addition, concrete filled steel structures have better fireproofing, soundproofing property than steel structures, and a considerable amount of time can be saved during the construction period.

The increased concern in concrete filled steel structures is attributed to the composite action of the two materials where the concrete core adds stiffness to steel shell by reducing the potential for inward local buckling and steel shell provides lateral confinement for the concrete core increasing the compressive strength approximately in (1.5 2) times as compared to the same grade of concrete without confinement. Many different cross sections shapes of concrete filled steel structures have been used, but the most commonly practical so far being concrete-filled steel tubular (CFST) columns, widely used in bridges and buildings (Morino, 1998; Shams, et al., 1997; Kitada, 1998; Roeder, 1998).

The behavior of composite columns has been the subject of extensive investigation since the beginning of the twentieth century. In 1915, Swain and Holmes studied the elastic behavior and strength of concrete-pipe columns. Kloppel and Goder (1957) carried out tests on the collapse load of CFST columns with different slenderness ratios. Basu and Sommerville in 1969 developed a design method of columns having different cross-sections and slendernesses. During last two decades, a number of experimental and numerical studies have been performed on the CFST columns (Nagashima, 1992; Chai, 1992; Boyd et al., 1995;Hunaiti, 1993; Liang and Uy, 2000; Huang et al., 2002; *Yinghua zhao2005;* Zhi-wu Yu,2007; Xu Kai-Cheng,2011). In Jordan, (Hunaiti et al., 1994; Shehdeh Ghannam et al., 2011) investigated experimentally, the behavior of partially encased composite columns subjected to eccentric load and the CFST columns with normal and lightweight concrete under axial loadings. Al - Dabayba (2000), conducted a comparative study on CFST columns and concluded that the codes used different design procedures for designing CFST columns and some differences were observed in numerical results. Bassam Z et al., (2002) investigated experimentally the effect of confinement in CFST columns and found that the increase in ductility of confined concrete is related to the stiffness properties of the confining steel.

Several design equations have been developed to find out the ultimate axial capacity of CFST columns (Gardner et al., 1967; Furlong, 1968; Knowles et al., 1970),Rangan and Joyce 1992). In the proposed equations the confinement effect of the steel tube on the concrete core was ignored. As a consequence, a close agreement between test results and the predicted ultimate capacities was not achieved. Schneider (1998) investigated the effect of steel tube and wall thickness on the ultimate strength of CFST columns. Different approaches giving significant discrepancies in results (Manojkumar, et al., 2010; Gupta, et al., 2007 Muhammad et al., 2006; O'Shea, et al., 2000; Shams, et al., 1997; Elnashai, et al., 1995; Zhang, et al, 1999) are currently being used for the estimation of the ultimate strength load of composite columns. Presently, the paper mainly concentrated on the predicting the ultimate axial load capacity (UALC) of CFST short columns with normal weight concrete and comparison with test results and the code predicted ultimate axial strength using the Euro code EC 4 and the Chinese CECS code specification.

RESEARCH SIGNIFICANCE

The main problem has to do with the fact that the state of stress that exist in the CFST columns are rather complex. The concrete core is under a tri-axial compound state of stress. The confining pressure that is developed between the concrete core and the steel tube interface varies at different stages of loading. As a result, the overall response of CFST columns under static loadings is not well understood due to lack of knowledge on the behavior of its constituent components and the interaction between them. The theoretical research herein consider the effect of important parameters, such as the variable value of enhanced compressive strength of concrete core with different concrete grades, the type or the failure mode of concrete core at ultimate state of stress, the variable direction of slip planes and the effect of confining lateral pressure on the concrete core expressed through the coefficient value of lateral confinement.

THE UALC OF CFST COLUMNS IN CODES DESIGN SPECIFICATION

Researchers have different interpretation of the (UALC) of CFST short columns. Cai [1] used the maximum load capacity attained as UALC. Han [2] defined the UALC as the load when a certain axial strain limit is reached. Miao [3] defined the UALC as the load when a cut through crack is formed, which is difficult to measure. In this research, the UALC is defined as the maximum load when the axial strain (ε) reaches yielding strain (ε_y), at the same time, the confined concrete core reaches its ultimate strength capacity.

The UALC of CFST columns can be determined using several methods available and the current in design codes. In brief, some of these methods are illustrated here [4].

I- The Euro code EC 4 specification

The *EC 4* procedures [5] are based primarily on the work of Roik and Bergman (1992).The UALC of of CFST:

$$N_u = \chi\left[\eta_2 A_a f_a + \left(1+\eta_1 \frac{t}{D}\frac{f_a}{f_c}\right)A_c f_c\right] \tag{1}$$

where $\eta_1$and η_2 are factors considering confinement effect, determined by the relative slenderness, λ .

$$\begin{cases}\eta_1 = 4.9 - 18.5\bar{\lambda} + 17\overline{\lambda^2} \\ \eta_2 = 0.25(3+2\bar{\lambda})\end{cases} \tag{2}$$

$$\bar{\lambda} = \sqrt{N_{pl,R}/N_{cr}} \;\; \leq 0.5 \tag{3}$$

where $N_{pl,R}$ is the plastic strength of the composite column calculated by

$$N_{pl,R} = f_a A_a + f_c A_c \tag{4}$$

and the N_{cr} is defined as the Euler buckling strength of the composite column.

$$N_{cr} = \frac{\pi^2 (EI)_e}{l^2}$$

$(EI)_e = A_a E_a + 0.6 A_c E_{sm}$ with E_{sm} is the secant modulus of concrete.
The buckling strength reduction factor χ is used to reduce the plastic compressive resistance of the cross section of slender columns.

$$\chi = \frac{1}{\left(\phi + \sqrt{\phi^2 - \bar{\lambda}^2}\right)} \leq 1 \tag{5}$$

where $\phi = 0.5\left[1 + 0.21\left(\bar{\lambda} - 0.2\right) + \overline{\lambda^2}\right]$ (6)

II-The Chinese CECS code specification.

The Chinese CECS code [6] is depend on unified theory and unified designing formula developed by Harbin University. The UALC of CFST columns is calculated by

$$N_u = \varphi_l \varphi_e N_0 \tag{7}$$

where N_0 the UALC of short CFST columns

$$\begin{cases} \varphi_l = 1 & \text{for } (l_e/D) \leq 4 \\ \varphi_l = 1 - 0.115\sqrt{l_e/D - 4} & \text{for } (l_e/D) > 4 \end{cases} \tag{8}$$

The φ_l and φ_e are reduction factors consider the eccentric loading effect and slenderness influence, respectively. For concentric loading, φ_e =1, and l_e is the effective length of the column, which is determined by the supporting conditions.

$$N_0 = f_c A_c + f_a A_a + \sqrt{(f_c A_c)(f_a A_a)} \tag{9}$$

where f_c =characteristic cylinder compressive strength of concrete; A_c = area of the concrete section; f_a = yield strength of steel; and A_a =Area of the steel section. The CECS considers the confinement effect by $\sqrt{(f_c A_c)(f_a A_a)}$.

As reviewed above, that the approach and also the noticeable differences in the design philosophies for calculating the capacity of CFST columns between the two Codes indicate that there is a need for a more practical and new design technique to predict the UALC of CFST columns.

THEORETICAL BASE FOR PREDICTING THE UALC OF CFST SHORT COLUMNS

Various elementary ideas of plastic deformation and failure , such as the yield surfaces, flow rules and slip lines ,emerged throughout the nineteenth century in the studies of pioneers such as Luders (1854), Tresca (1868), de St. Vénant (1870), Lévy (1870), Rankine (1876), Bauschinger (1881), Considère (1891), Engesser (1895), Hartmann (1896) , Mohr (1900) von Kármán (1909), von Mises (1913)and Hencky (1924) [7]. Von Kármán and others of researchers interested in the direction of angles formed by slip planes with respect to the major principal compressive stress. These angles increased in plastic marble from 53 deg with no hydraulic pressure to

73 deg with a hydraulic pressure of 686 atm. Nadai [7] assert that there is a close connection between the orientation of these sliding planes and the structure of polycrystalline solids. It is well known that specimens of brittle crystalline materials, such as natural rocks, cast iron, or of certain brittle conglomerates of materials (concrete), in the ordinary axial compression test, two sets of slip lines intersecting with constant angle, will be generated and break along surfaces obliquely inclined with respect to the direction of compressive stress . These fracture surfaces are inclined at an angle always smaller than 45° with respect to the direction of compression. A model proposed by Kim and Mander [8] estimates the crack angle based on minimizing the external work due to a unit shear force. For the concrete columns tested by Lynn and Sezen[9], the critical crack angle estimated by the model ranges from 65°to71° degrees, with an average of 68° relative to horizontal plane. Seminenko .I.P[10] suggested determining the angle of shear failure plane Ø with respect to horizontal plane of unconfined normal weight concrete specimen subjected to axial compression by:

$$\phi = \tan^{-1}(\frac{1}{\rho}(1+\sqrt{1-\rho}) \qquad (10)$$

where ρ =ratio of the characteristic prismatic strength f_{pr} with a height to width ratio greater than 2 to characteristic cubic strength f_{cu} of normal weight concrete specimen of the same material constituents and cross sectional dimensions .

The behavior of concrete in CFST columns becomes more ductile with lateral confining pressure $\sigma_2=\sigma_3$. The axial compressive strength σ_1 and the corresponding strain is higher than those of unconfined concrete. From the experimental results, an attempt is made to predict the confined strength of the different fill materials by employing many proposed relations, which relate the confined strength to the unconfined strength and lateral confining stress. The analytical determination of radial stresses $\sigma_2=\sigma_3$ on concrete core is still uncertain, but some relations have been recommended from experimental data relating the radial stress to the steel tube dimensions and yield strength. The value of lateral confining pressure determined in this research is corresponding to the maximum concrete stress, and the steel shell strains at this point around the yield strain. Hence, the peak stress of the confined concrete for CFST columns corresponds to high lateral pressure and can be expressed as:

$$\sigma_2=\sigma_3=k_0\sigma_1 \cong max \qquad (11)$$

k_0 = coefficient of lateral pressure.

The radial stresses $\sigma_2=\sigma_3$ on concrete core can be defined at complete lack of transverse displacements ($\Delta u,\Delta v$) of steel shell:

$$\Delta u = \Delta v = 0 \tag{12}$$

And, as a corollary, the transverse strains:

$$\varepsilon_2 = \varepsilon_3 = 0 \tag{13}$$

In the proposed approach, k_0 in (11) can be determined as suggested by Rydakov.B.N and the author [11]:

(14)

As a result, $\phi \approx 0$ and $k_0 \approx 1$ for an ideal liquid body. $\phi \approx 90°$ and $k_0 \approx 0$, for an undeformable rigid body. Follows:

$0 < k_0 < 1$

For normal weight concrete with characteristic cylindrical compressive strength $f_c^{'}$=12 Mpa, k_0 =0.191; and with $f_c^{'}$=40 Mpa k_0 =0.156. Gradually increasing the concrete strength will decrease the value of k_0. Attard *et al.* (1996) performed a test series of high-strength concrete subjected to low confining pressure and Ansari and Li (1998) carried out a comprehensive experimental program with high confining pressure. They found that the influence of confining pressure on the maximum compressive strength of high-strength concrete is not so pronounced as on that of normal strength concrete.

It is possible to estimate the ultimate strength load of concrete core by applying an appropriate strength criterion [12] at $\sigma_2 = \sigma_3 = k_0\sigma_1 \cong max$. The proposed strength criterion below attempts to consider the possible shear failure mode of concrete core and the state of stresses acting on the inclined shear failure plane at angle ϕ:

$$f_k = f_{pr}\sqrt{\frac{\epsilon}{f_t}} - f_0 \tag{15}$$

where f_k =ultimate axial compressive strength; ϵ =equivalent tensile strength; f_t =tensile axial strength of concrete; f_0 =average pressure acting on concrete core.

By substituting ϵ , f_t and f_0 into Eq. (15), setting $\epsilon = kf_{pr} + f_t$; $f_t = 0.5 f_{pr} cos^2\phi$; and $f_0 = kf_k$

We obtain;

$$f_k = f_{pr}\sqrt{\frac{kf_{pr} + 0.5f_{pr}cos^2\phi}{0.5f_{pr}cos^2\phi}} - kf_k \tag{16}$$

and solving for f_k , we obtain an estimation of the ultimate strength capacity of concrete core as a function of k, ϕ and f_{pr}

$$f_k = \frac{k f_{pr}^2 + f_{pr}\sqrt{k^2 f_{pr}^2 + (f_{pr}\cos^2\phi)^2(1+k)^2}}{f_{pr}\cos^2\phi(1+k)^2} \quad (17)$$

Therefore, the UALC of CFST short columns stipulated by the concept of shear failure plane direction can be proposed in such form:

$$P_u = A_c f_k + A_s f_y \quad (18)$$

where P_u = the UALC of CFST short columns; A_c =cross-sectional area of a concrete core.

A_s =cross-sectional area of steel tube; and f_y =nominal yield strength of steel tube.

COMPARISON WITH DESIGN CODES AND TEST DATA

IN order to verify the proposed formula with the experimental tests and design codes, namely EC4 and CECS for predicting the UALC of the CFST short columns subjected to concentric loading, a total of 100 test data with the geometric and material properties of the tests specimens reported in Table 1 extracted from [4] are adopted. The UALC predictions of all the 100 tests by proposed formula as well as the EC4 and the CECS methods are listed in Table 1. The reported cubic concrete strength (f_{cu}) values in Table 1 were converted to the cylindrical strength for normal weight (f_c') according to the Neville's expression [13] as follow:

$$f_c' = \left[0.76 + 0.2\log\left(\frac{f_c'}{19.58}\right)\right] f_{cu} \qquad \text{(SI units)} \quad (19)$$

On the basis of statistical handling of data, the prismatic concrete strengths were converted to the cubic concrete strength for normal weight by the expression [1]:

$$f_{pr} = \frac{1}{3} f_{cu}\left(\frac{288 + f_{cu}}{110 + f_{cu}}\right) \qquad \text{(SI units)} \quad (20)$$

The predictions by all the methods agree well with the test data. The standard error deviations by using the current method, the EC4 method and the CECS method are 0.1202, 0.1205 and 0.1606, respectively. The predictions by the current method and the EC4 method are close to each other. The error predicted by the CECS seems a bit greater. The CECS tends to overestimate the UALC for concrete columns. The comparison shows that the current method is a good alternative to the other methods and gives very good prediction of the UALC of the CFST tested columns.

The following three steps have been applied to find the ultimate strength of normal weight concrete core by proposed method.The results are shown in Table 1:

Step1. Calculating the direction of shear failure plane with relative to horizontal plane of unconfined normal weight concrete specimen by (10) or using the graphical plot (Fig. 1).

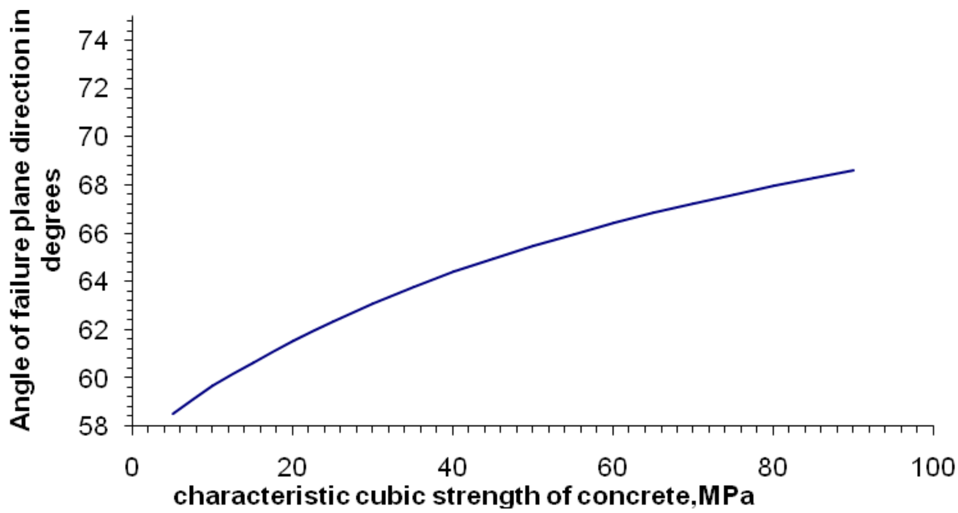

Figure.1.Characteristic cubic strength of concrete versus the direction of failure plane

Step2. Determining the coefficient of lateral pressure *k* of confined normal weight concrete as a function of Ø by (10) or using the graphical plot (Fig. 2).

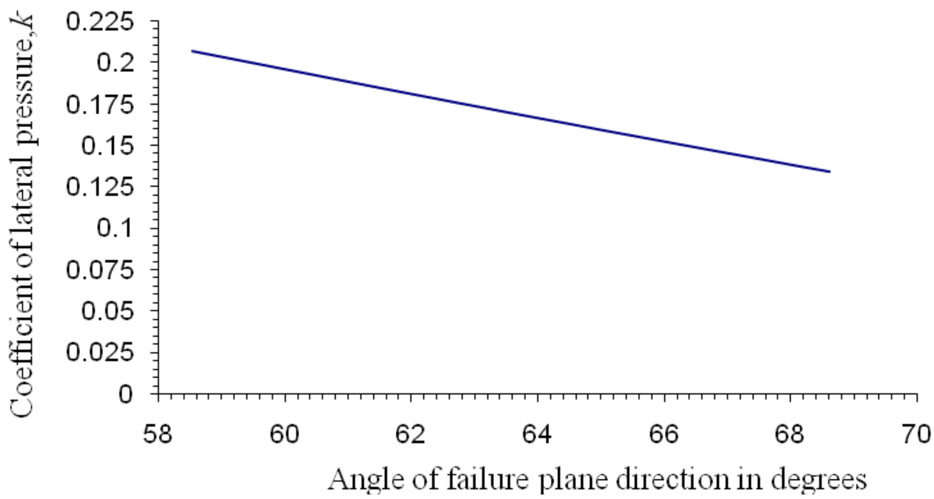

Figure2. Relation between the coefficient of lateral pressure and the direction of failure plane

Step 3.Depending on the value of the coefficient lateral pressure *k* and the nominal compressive strength of unconfined normal weight concrete and the direction of failure plane of unconfined concrete Ø we determine the value of strengthening concrete core f_k by(17) .

TABLE 1
THE UALC OF THE CFST SHORT COLUMNS BY THE PROPOSED METHOD, THE EC4 AND THE CECS METHODS
(P_t=test data; P_u = UALC calculated using proposed method; Pec4= UALC calculated using the EC4 method; Pce= UALC calculated using the CECS method)

NO.	L (mm)	D (mm)	t (mm)	f_y (MPa)	f_c' (MPa)	P_t (kN)	f_{cu} (MPa)	f_k (MPa)	P_u (kN)	$\frac{P_u-P_t}{P_t}$	P_{ec4} (kN)	$\frac{P_{ec4}-P_t}{P_t}$	P_{ce} (kN)	$\frac{P_{ce}-P_t}{P_t}$
1	305	168.3	3.60	288.40	27.00	1557	33.4	42.1	1394	-0.105	1429	-0.08	1630	0.047
2	305	168.3	3.60	288.40	33.30	1432	42.3	53.1	1618	0.130	1551	0.08	1819	0.270
3	305	168.3	3.60	288.40	33.30	1463	42.3	53.1	1618	0.106	1551	0.06	1819	0.243
4	229	114.3	3.50	350.00	33.40	969	42.4	53.3	908	-0.063	971	0.00	1087	0.122
5	229	114.3	4.50	339.00	33.40	1069	42.4	53.3	990	-0.074	1107	0.04	1208	0.130
6	360	178.0	9.00	283.00	22.20	2120	27.6	34.9	2053	-0.032	2541	0.20	2574	0.214
7	360	178.0	9.00	283.00	22.20	2060	27.6	34.9	2053	-0.003	2541	0.23	2574	0.250
8	360	178.0	9.00	283.00	45.40	2720	55.4	69.2	2743	0.008	2976	0.09	3374	0.240
9	360	178.0	9.00	283.00	45.40	2730	55.4	69.2	2743	0.005	2976	0.09	3374	0.236
10	360	179.0	5.50	249.00	22.10	1410	27.5	34.8	1516	0.075	1691	0.20	1840	0.305
11	360	179.0	5.50	249.00	23.90	1560	29.7	37.4	1576	0.010	1729	0.11	1904	0.221
12	360	179.0	5.50	249.00	43.70	2080	53.7	67.1	2234	0.074	2144	0.03	2564	0.233
13	360	179.0	5.50	249.00	43.70	2070	53.7	67.1	2234	0.079	2144	0.04	2564	0.239
14	360	174.0	3.00	266.00	23.90	1220	29.7	37.5	1258	0.031	1225	0.00	1434	0.175
15	360	174.0	3.00	266.00	23.90	1220	29.7	37.4	1258	0.031	1225	0.00	1434	0.175
16	477	159.0	5.07	381.50	41.50	2230	51.5	64.4	2056	-0.078	2049	-0.08	2478	0.111
17	1890	630.0	8.44	350.00	34.50	18600	44.2	55.4	22127	0.190	18588	0.00	23607	0.269
18	1890	630.0	10.21	323.30	38.40	20500	48.4	60.6	24107	0.176	20535	0.00	26108	0.274
19	1890	630.0	11.60	347.20	46.00	24400	56.0	70.0	28042	0.149	24475	0.00	31314	0.283
20	2160	720.0	8.30	312.00	15.00	15000	18.8	23.8	15018	0.001	14739	-0.02	17419	0.161
21	264	131.8	2.38	235.00	17.40	535	21.8	27.5	576	0.077	596	0.11	672	0.256
22	264	134.3	3.12	235.00	26.60	681	32.9	41.5	836	0.228	834	0.22	966	0.419

23	264	130.6	4.30	235.00	26.60	725	32.9	41.5	885	0.221	952	0.31	1064	0.468
24	264	132.5	5.25	235.00	26.60	872	32.9	41.5	978	0.121	1094	0.25	1195	0.370
25	264	134.1	6.20	235.00	26.60	1006	32.9	41.5	1067	0.061	1231	0.22	1320	0.312
26	200	101.8	2.94	320.00	18.00	628	22.5	28.5	498	-0.207	600	-0.04	617	-0.018
27	200	101.8	2.94	320.00	37.40	660	47.4	59.4	721	0.092	731	0.11	843	0.277
28	200	101.8	5.70	305.00	37.40	971	47.4	59.4	906	-0.067	1039	0.07	1119	0.152
29	200	100.0	0.52	244.00	18.00	239	22.5	28.5	259	0.082	205	-0.14	252	0.054
30	270	86.5	2.73	226.70	30.20	412	37.3	46.9	405	-0.018	396	-0.04	478	0.160
31	270	89.3	4.00	226.70	30.20	491	37.3	46.9	486	-0.010	513	0.04	594	0.210
32	270	86.5	2.79	226.70	48.00	489	58.0	72.4	539	0.102	487	0.00	616	0.260
33	270	89.2	4.05	226.70	48.00	605	58.0	72.4	619	0.024	600	-0.01	740	0.223
34	266	76.0	2.20	390.00	57.00	470	70.2	87.3	550	0.171	495	0.05	642	0.366
35	266	76.0	2.20	390.00	57.00	420	70.2	87.3	550	0.310	495	0.18	642	0.529
36	266	76.0	2.20	390.00	57.00	465	70.2	87.3	550	0.183	495	0.06	642	0.381
37	356	101.7	2.40	380.00	57.00	770	70.2	87.3	928	0.205	800	0.04	1050	0.364
38	356	101.7	2.40	380.00	57.00	775	70.2	87.3	928	0.198	800	0.03	1050	0.355
39	356	101.7	2.40	380.00	57.00	740	70.2	87.3	928	0.254	800	0.08	1050	0.419
40	356	101.7	2.40	380.00	57.00	775	70.2	87.3	928	0.198	800	0.03	1050	0.355
41	581	165.0	2.82	363.30	48.30	1662	58.3	72.8	1973	0.187	1669	0.00	2193	0.319
42	664	190.0	1.94	256.40	41.00	1678	51.0	63.8	2029	0.209	1523	-0.09	1981	0.181
43	310	105.1	2.85	264.90	18.40	550	23.0	29.1	468	-0.149	511	-0.07	571	0.038
44	310	107.9	4.32	264.90	18.40	686	23.0	29.1	597	-0.129	700	0.02	745	0.086
45	424	107.9	4.32	264.90	18.40	727	23.0	29.1	597	-0.178	662	-0.09	745	0.025
46	424	107.9	4.32	264.90	18.40	734	23.0	29.1	597	-0.186	662	-0.10	745	0.015
47	470	153.9	1.80	356.10	18.40	981	23.0	29.1	822	-0.162	788	-0.20	948	-0.034
48	470	155.6	2.63	356.10	23.00	1300	28.6	36.1	1090	-0.161	1077	-0.17	1286	-0.011
49	470	159.3	5.25	356.10	21.90	1577	27.3	34.5	1503	-0.047	1706	0.08	1872	0.187
50	470	160.2	5.40	356.10	21.90	1775	27.3	34.5	1538	-0.133	1754	-0.01	1917	0.080
51	470	159.8	5.08	356.10	21.90	1746	27.3	34.5	1484	-0.150	1676	-0.04	1845	0.057

52	812	264.6	4.55	323.40	21.90	3579	27.3	34.5	2967	-0.171	2915	-0.19	3485	-0.026
53	812	265.0	4.75	323.40	21.90	3789	27.3	34.5	3021	-0.203	2995	-0.21	3565	-0.059
54	812	264.4	4.50	323.40	21.90	3357	27.3	34.5	2952	-0.121	2894	-0.14	3463	0.032
55	399	111.3	2.00	354.60	42.70	840	52.6	65.9	839	-0.001	717	-0.15	936	0.114
56	337	113.6	3.20	354.60	42.70	1141	52.6	65.9	988	-0.134	949	-0.17	1168	0.024
57	338	113.6	3.20	354.60	42.70	1091	52.6	65.9	988	-0.094	949	-0.13	1168	0.071
58	336	113.6	3.20	354.60	42.70	1139	52.6	65.9	988	-0.133	949	-0.17	1168	0.025
59	335	114.8	3.90	357.70	42.70	1041	52.6	65.9	1078	0.036	1079	0.04	1301	0.250
60	338	114.8	3.90	357.70	42.70	1110	52.6	65.9	1078	-0.029	1078	-0.03	1301	0.172
61	343	114.8	3.90	357.70	42.70	1030	52.6	65.9	1078	0.047	1075	0.04	1301	0.263
62	356	115.9	4.90	309.50	42.70	1122	52.6	65.9	1111	-0.010	1126	0.00	1353	0.206
63	344	115.9	4.90	309.50	42.70	1234	52.6	65.9	1111	-0.100	1132	-0.08	1353	0.096
64	340	115.9	4.90	309.50	42.70	1102	52.6	65.9	1111	0.008	1134	0.03	1353	0.228
65	357	115.9	4.90	309.50	42.70	1140	52.6	65.9	1111	-0.025	1126	-0.01	1353	0.187
66	396	130.1	2.30	324.30	42.70	1240	52.6	65.9	1114	-0.101	958	-0.23	1225	-0.012
67	397	133.1	4.50	324.30	42.70	1440	52.6	65.9	1386	-0.037	1361	-0.05	1657	0.151
68	450	158.7	0.90	221.00	18.70	700	23.4	29.6	670	-0.043	516	-0.26	649	-0.073
69	450	157.5	1.50	308.00	18.70	815	23.4	29.6	780	-0.042	700	-0.14	858	0.053
70	450	157.7	2.14	286.00	18.70	908	23.4	29.6	845	-0.069	807	-0.11	966	0.064
71	1100	273.0	8.00	306.70	29.62	5576	36.5	46.0	4425	-0.206	4306	-0.23	5243	-0.060
72	1100	273.0	8.00	306.70	40.28	5194	50.3	62.9	5305	0.021	4807	-0.07	6073	0.169
73	1100	273.0	8.00	306.70	40.28	5292	50.3	62.9	5305	0.002	4807	-0.09	6073	0.148
74	465	133.0	3.50	352.00	106.02	1995	121.2	149.2	2361	0.183	1954	-0.02	2636	0.321
75	465	133.0	3.50	352.00	106.02	1991	121.2	149.2	2361	0.186	1954	-0.02	2636	0.324
76	465	133.0	4.70	352.00	106.02	2273	121.2	149.2	2456	0.081	2123	-0.07	2858	0.257
77	465	133.0	4.70	352.00	106.02	2158	121.2	149.2	2456	0.138	2123	-0.02	2858	0.324
78	465	133.0	4.70	352.00	106.02	2253	121.2	149.2	2456	0.090	2123	-0.06	2858	0.269
79	445	127.0	7.00	429.00	106.02	3370	121.2	149.2	2627	-0.220	2434	-0.28	3291	-0.023
80	990	219.0	7.00	273.00	38.20	3278	48.2	60.4	3264	-0.004	2913	-0.11	3483	0.063

81	990	219.0	7.00	273.00	38.20	3278	48.2	60.4	3264	-0.004	2913	-0.11	3483	0.063
82	990	219.0	7.00	273.00	38.20	3278	48.2	60.4	3264	-0.004	2913	-0.11	3483	0.063
83	1200	219.0	7.00	273.00	38.20	3200	48.2	60.4	3264	0.020	2790	-0.13	3267	0.021
84	1200	219.0	7.00	273.00	38.20	3200	48.2	60.4	3264	0.020	2790	-0.13	3267	0.021
85	1200	219.0	7.00	273.00	38.20	3200	48.2	60.4	3264	0.020	2790	-0.13	3267	0.021
86	1420	219.0	7.00	273.00	38.20	3070	48.2	60.4	3264	0.063	2670	-0.13	3110	0.013
87	1420	219.0	7.00	273.00	38.20	3070	48.2	60.4	3264	0.063	2670	-0.13	3110	0.013
88	1420	219.0	7.00	273.00	38.20	3070	48.2	60.4	3264	0.063	2670	-0.13	3110	0.013
89	1640	219.0	7.00	273.00	38.20	2956	48.2	60.4	3264	0.104	2569	-0.13	2983	0.009
90	1640	219.0	7.00	273.00	38.20	2956	48.2	60.4	3264	0.104	2569	-0.13	2983	0.009
91	1640	219.0	7.00	273.00	38.20	2956	48.2	60.4	3264	0.104	2569	-0.13	2983	0.009
92	1420	95.0	3.50	348.88	26.20	582	32.4	40.9	599	0.030	504	-0.13	449	-0.229
93	1050	121.0	4.00	311.15	22.20	703	27.6	34.9	807	0.148	790	0.12	679	-0.034
94	1050	121.0	4.00	317.03	26.50	852	32.8	41.3	880	0.033	831	-0.02	722	-0.153
95	1050	121.0	6.00	349.37	22.20	1007	27.6	34.9	1082	0.075	1142	0.13	949	-0.058
96	1050	121.0	6.00	325.85	26.50	1089	32.8	41.3	1091	0.002	1122	0.03	940	-0.137
97	2220	216.0	6.00	391.02	24.10	2440	29.9	37.7	2780	0.139	2519	0.03	2169	-0.111
98	2220	216.0	6.00	379.26	31.40	2866	39.2	49.3	3112	0.086	2604	-0.09	2337	-0.185
99	2220	216.0	4.00	289.39	24.10	1869	29.9	37.7	2052	0.098	1588	-0.15	1506	-0.194
100	2220	216.0	4.00	287.14	31.40	2262	39.2	49.3	2439	0.078	1711	-0.24	1715	-0.242
							Average error			0.022		-0.03		0.135
							Standard deviation			0.1193		0.1205		0.1606
							Maximum error			0.310		0.313		0.529
							Minimum error			-0.220		-0.278		-0.242

CONCLUSIONS

It is shown that the predictions by this method agree well with the test results and those predicted by the EC4 and the CECS methods. One can see, moreover, that the ratios D/t =18 up to 192 and *L/D*=1.8 up to 10 of test data has insignificant effect on the results of suggested formula UALC of the CFST short columns. The enhanced strength of concrete core by the proposed approach approximately equals $(1.4\text{up to}1.6)\ f_c^{'}$ and $(1.23$ up to $1.27)f_{cu}$.

The proposed formula even yields reasonable prediction for higher concrete strength overtaking the code restriction requirements for the upper limits. In our belief, verification on the accurateness of proposed assumption formula should be confirmed with more test data.

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