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Reliability-based assessment of American and European specifications for square CFT stub columns

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Abstract. This paper presents a probabilistic investigation of American and European specifications (i.e., AISC and Eurocode 4) for square concrete-filled steel tubular (CFT) stub columns. The study is based on experimental results of 100 axially loaded square CFT stub columns from the literature. By comparing experimental results for ultimate loads with code-predicted column resistances, the uncertainty of resistance models is analyzed and it is found that the modeling uncertainty parameter can be described using random variables of lognormal distribution. Reliability analyses were then performed with/without considering the modeling uncertainty parameter and the safety level of the specifications is evaluated in terms of sufficient and uniform reliability criteria. Results show that: (1) The AISC design code provided slightly conservative results of square CFT stub columns with reliability indices larger than 3.25 and the uniformness of reliability indices is no better because of the quality of the resistance model; (2) The uniformness of reliability indices for the Eurocode 4 was better than that of AISC, but the reliability indices of columns designed following the Eurocode 4 were found to be quite below the target reliability level of Eurocode 4.

Keywords: composites column; concrete; steel; square tubes; specifications; modeling uncertainty parameter; reliability analysis

1. Introduction

Concrete filled steel tubular (CFT) columns are widely used in the construction of high-rise buildings, bridges, subway platforms, barriers and offshore structures, mainly because CFT columns combine the advantages of ductility, generally associated with steel structures, with the stiffness of a concrete structural system.

Two main shapes of CFT columns, i.e., circular and square cross-sectional tubes, are used in the practical engineering structures. Of interest here are square CFT columns and the typical cross-sections for square CFT columns are illustrated in Fig. 1, where B is the width of square steel tube and t is the wall thickness of the steel tube. Square CFT columns are used gradually more and more as one of the main structural elements for resisting both vertical and lateral loads due to their advantages compared to circular CFT columns such as (Cai and He 2006): (1) the

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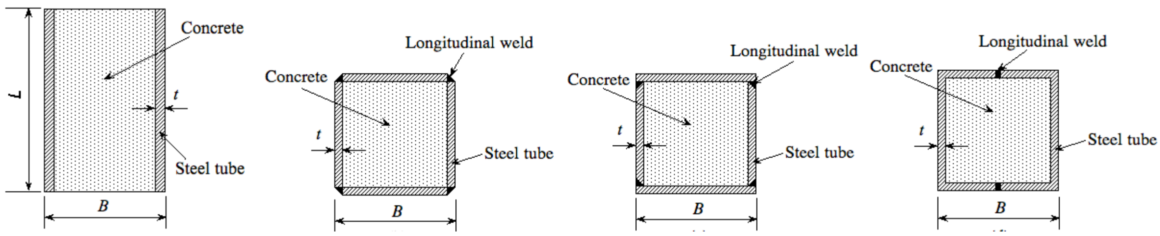


Fig. 1 Ilustration of square CFT columns and typical cross-sections for square CFT columns

cross-section shape agreeing well with the design need of the architectural plane; (2) more convenient construction measures at beam-column joints resulting in easy connection and less cost; (3) larger moment of inertia of cross section compared with the circular CFT of the same overall dimension which leads to higher capacity of resisting lateral load. Due to these advantages and increasing usage worldwide, research on square CFT columns has been ongoing worldwide for decades (e.g., Tomii and Sakino 1979, Ge and Usami 1992, Kato 1995, Uy 2000, Han and Tao 2001, Yamamoto *et al*. 2003, Lam and Williams 2004, Sakino *et al*. 2004, Liu 2005, Liu and Gho 2005, Huang *et al*. 2008, Uy *et al*. 2011, Chen *et al*. 2012, Evirgen *et al*. 2014). Almost all the researches focused on the strength evaluation of square CFT columns, the analysis of experimental results however is not sufficient and probabilistic investigation of design code provisions are seldom conducted.

A first-order second-moment reliability study performed by Lundberg and Galambos (1996) revealed that the concrete-encased members exceeded the target reliability index in the design specification, but that the CFT columns (circular tubes and rectangular tubes) had an inadequate reliability index when compared to the target value. Recently, Beck *et al*. (2009) presented a reliability-based evaluation of design code provisions for circular CFT columns. However, no studies have been conducted for reliability assessment of design codes for the square CFT columns.

This paper addresses the safety of American and European design code provisions, i.e., AISC (2005) and Eurocode 4 (2004), for square CFT stub columns. A brief review of the two design-code provisions is presented in Section 2. In Section 3 experimental results taken from the literature are briefly reviewed. Section 4 addresses the error of code resistance models, by comparing experimental results for ultimate loads with code-predicted column resistances. Reliability analysis and the evaluation of the two design code provisions are presented in Section 5. Finally, the main conclusions obtained from the present study are summarized in Section 6.

2. Design code provisions for square CFT stub columns

For completeness, a brief review of the determination of the axial capacity of square CFT stub columns using the methods described in the codes of AISC (2005) and Eurocode 4 (2004) is presented as follows.

2.1 The AISC (2005)

In the AISC (2005) design code, the design resistance of a CFT column is given by

$$N_{RD} = \phi_c P_n \tag{1}$$

where ϕ_c is the partial factor applied to column resistance (= 0.75). The nominal resistance P_n is obtained from Eq. (2)

$$P_n = \begin{cases} P_{0,AISC}\left[0.658^{\left(\frac{P_{0,AISC}}{P_e}\right)}\right] & P_e \geq 0.44 P_{0,AISC} \\ 0.877 P_e & P_e < 0.44 P_{0,AISC} \end{cases} \tag{2}$$

where $P_{0,AISC}$ is the capacity of the cross section (zero length strength) and P_e is the elastic buckling load. For the prediction of $P_{0,AISC}$ in the AISC approach, the strength enhancement of core concrete due to confinement by steel hollow section was omitted. A coefficient of 0.85 is included in the cylinder strength of concrete to account for long-term and size effects. Therefore, the cross-sectional strength, $P_{0,AISC}$ is given by

$$P_{0,AISC} = 0.85 f_{cyl,150} A_c + f_y A_s \tag{3}$$

where A_c = the cross-sectional area of the concrete; A_s = the cross-sectional area of the steel tube; $f_{cyl,150}$ = the concrete compressive strength obtained from cylinder test of 150 X 300 mm specimen; and f_y = the yield strength of the steel tube.

According to AISC (2005), P_e is given by

$$P_e = \frac{\pi^2 (EI)_{eff}}{(K_A L_A)^2} \tag{4}$$

in which

$$(EI)_{eff} = E_s I_s + C_3 E_c I_c \tag{5}$$

$$C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.9 \tag{6}$$

K_A = the effective length factor; L_A = laterally unbraced length of the column; I_s and I_c = moment of inertia of steel tube and concrete core, respectively; E_s = Modulus of elasticity of steel = 200000 MPa; E_c = the modulus of elasticity of concrete = 4730 $(f_{cyl,150})^{1/2}$ MPa (normal weight concrete); and $(EI)_{eff}$ = effective stiffness of composite section.

2.2 The Eurocode 4 (2004)

Eurocode 4 (2004) is the most recent international standard in composite construction. Eurocode 4 (2004) covers concrete encased and partially encased steel sections and concrete filled sections with or without reinforcement. According to Eurocode 4 (2004), the axial ultimate capacity of a square CFT stub column can determined by summing up the yield load of the steel section and that of the concrete core. The coefficient of 0.85, which applies to reinforced concrete columns, is set to 1.0 to incorporate the beneficial confining effect, as shown in Eq. (7).

Table 1 Partial safety coefficients for loads and resistances, according to code design

Code	Resistance			Load combination (Dead + Live)
	Steel (γ_a)	Concrete (γ_c)	Member (ϕ_c)	$\gamma_D \cdot Dead + \gamma_L \cdot Live$
AISC (2005)	-	-	0.75	1.4*Dead* 1.2*Dead* + 1.6*Live*
Eurocode 4 (2004)	1.1	1.5	-	1.35*Dead* + 1.5*Live*

$$N_{RD} = \frac{f_{cyl,150} A_c}{\gamma_c} + \frac{f_y A_s}{\gamma_a} \tag{7}$$

where γ_a, γ_c = the partial resistance factors for steel and concrete.

2.3 The design resistance and actual resistance

The load and resistance factors used in the two design code provisions described above are shown in Table 1. Partial resistance factors and characteristic resistance values are used in normal design situations, when a safety margin is to be created. When the actual (model) resistance is required, the safety margin is eliminated by setting partial factors to unity and by replacing characteristic values of material resistance by mean values. In order to distinguish the two situations, the design resistance (with safety margins) is denoted by N_{RD} and actual resistance (without safety margins and with mean material values) is denoted by N_{RA}. The actual resistance is required in two instances in this paper: first, when comparing design code (model) resistances with experimental results (Section 4); second, in reliability analysis and safety evaluation (Section 5).

3. Experimental results from the literature

In order to obtain a complete picture of design code provisions for square CFT columns, 100 experimental results from the literature were considered in this study. Liu (2005) tested 8 square CFT columns with wall thicknesses of t = 4 mm. These columns had slenderness ratios L/B = 3 (L is the lengths of the columns), for tube width of B = {106, 120, 130and 140} mm. The internal concrete had average cylinder strengths of 60 and 89 MPa and the average yield strength of steel tube was determined as 495 MPa.

The specimens used in the tests conducted by Liu and Gho (2005) had tube width of B = {120, 130 and 200} mm and wall thicknesses of t = {4 and 5.8} mm. The test specimens were short with a length-to-width (L/B) ratio of 3.0. The internal concrete had average cylinder strengths of 50, 83, and 106 MPa and the average yield strengths of steel tube were determined as 300 and 495 MPa.

Tomii and Sakino (1979) tested 8 CFT columns, with a width B = 100 mm and a length of 300 mm, resulting in an L/B ratio of 3.0. Columns were filled with concrete cylinder strengths of 31.9, 21.4, 20.6 and 19.8 MPa and the yield strengths of steel tube f_y = {194, 339.1, 288.1 and 284.2} MPa. To ensure the width-to-thickness (B/t) ratios of the specimens are below the limit values specified in the design codes of AISC and Eurocode 4, wall thicknesses of t = {2.29 and 4.25} mm were considered. 6 results by the authors were incorporated in this study.

Kato (1995) tested 12 square CFT columns, but only 6 had wall thicknesses within acceptable limits for the design codes considered herein t = {7.65 and 11.9} mm. These columns had slenderness ratios L/B = 3, for tube width of 250 mm. The internal concrete had cylinder strengths of 30.1, 35.6, and 82.4 MPa and the yield strengths of steel tube were determined as 316.5 and 495 MPa.

Twenty-three specimens of square CFT columns subjected to axial load examined by Yamamoto *et al*. (2003) had the nominal width of 100, 200, 300 and 400 mm and corresponding nominal tube plate thicknesses of 2.2, 3.0, 5.8, 9.0 and 12.5 mm. The lengths of the columns were three times of the width. The nominal design concrete cylinder strengths were 26.5, 47, and 64.7 MPa and the yield strength of steel tube were determined as 300, 330, 375, 399 and 455 MPa. In view of the width-to-thickness (B/t) ratios within acceptable limits for the design codes considered, 10 results by the authors were incorporated in this study.

Sakino *et al*. (2004) tested 48 square CFT columns with slenderness ratios L/B = 3, but only 36 had width-to-thickness (B/t) within acceptable limits for the design codes considered. The main parameters varied in these columns are: (1) the width of the steel tube (B) from 119 to 324 mm; (2) the wall thickness (t) from 4.38 to 9.45; (3) the yield strength of steel tube form 262 to 835 MPa; and (4) design concrete cylinder strength from 25.4 to 91.1 MPa.

Han and Tao (2001) tested 20 columns, with tube width of B = {120, 140 and 200} mm and wall thicknesses of t = {3.8 and 5.9} mm. The lengths of the columns were three times of the width. The design concrete cube strengths varied from 17.6 to 54.6 MPa and the yield strength of steel tube were determined as 321.1 and 330.1 MPa.

Ten square CFT columns were loaded uniformly over the concrete and the steel section by Lam and Williams (2004). All specimens were 300 mm in length and the nominal tube width were 100 mm with the wall thicknesses of t = {4.0, 5.0 and 10.0} mm. The design concrete cube strengths varied from 30.8 to 98.9 MPa and the yield strength of steel tube were determined as 289, 333 and 400 MPa.

The details of the 100 experimental columns such as the measured dimensions, material properties, and the ultimate strength were summarized in Table 2, in which $f_{cyl,100}$ corresponds to the concrete's compressive strength obtained from cylinder tests of 100 × 200 mm, $f_{cu,150}$ denotes the concrete compressive strength with 150 mm cube tests and the yield stress of steel tube f_y is obtained from tensile coupon tests (for the case of the steel having no clear yield plateau, it

Table 2 Measured specimen dimensions, material properties, axial capacities, and modeling uncertainty parameter (M) statistics of square CFT stub columns

No. of specimens	Name of specimens	Dimensions			Material properties		Axial capacity N_{test}(kN)	Tested by	$M = N_{test}/N_{RA}$	
		B (mm)	t (mm)	L (mm)	f_c (MPa)	f_y (MPa)			AISC	Eurocode 4
1	R1-1	120	4	360	60	495	1701	Liu (2005) (8 tests)	1.091	1.018
2	R1-2	120	4	360	60	495	1657		1.063	0.991
3	R4-1	130	4	390	60	495	2020		1.150	1.068
4	R4-2	130	4	390	60 ($f_{cyl,150}$)	495	2018		1.149	1.067
5	R7-1	106	4	320	89	495	1749		1.140	1.052
6	R7-2	106	4	320	89	495	1824		1.189	1.097
7	R10-1	140	4	420	89	495	2752		1.149	1.047
8	R10-2	140	4	420	89	495	2828		1.181	1.076

Table 2 Continued

No. of specimens	Name of specimens	Dimensions			Material properties		Axial capacity	Tested by	$M = N_{test}/N_{RA}$	
		B (mm)	t (mm)	L (mm)	f_c (MPa)	f_y (MPa)	N_{test}(kN)		AISC	Eurocode 4
9	A1	120	5.8	360	83	300	1697	Liu and Gho (2005) (8 tests)	1.045	0.959
10	A2	120	5.8	360	106	300	1919		1.035	0.941
11	A3-1	200	5.8	600	83	300	3996		1.036	0.930
12	A3-2	200	5.8	600	83 ($f_{cyl,150}$)	300	3862		1.002	0.899
13	A9-1	120	4	360	55	495	1739		1.155	1.081
14	A9-2	120	4	360	55	495	1718		1.141	1.068
15	A12-1	130	4	390	55	495	1963		1.159	1.081
16	A12-2	130	4	390	55	495	1988		1.174	1.094
17	I-A	100	2.29	300	31.9	194	497	Tomii and Sakino (1979) (6 tests)	1.215	1.102
18	I-B	100	2.29	300	31.9	194	498		1.217	1.104
19	III-A	10	2.99	300	20.6 ($f_{cyl,100}$)	288.1	528		1.106	1.049
20	III-B	100	2.99	300	20.6	288.1	527		1.104	1.047
21	IV-A	100	4.25	300	19.8	284.2	666		1.134	1.090
22	IV-B	100	4.25	300	19.8	284.2	665		1.132	1.088
23	R08LB	250	7.65	750	30.1	358.7	4655	Kato (1995) (6 tests)	1.160	1.095
24	R12LB	250	11.9	750	30.1	316.5	5635		1.164	1.113
25	R08MB	250	7.65	750	35.6 ($f_{cyl,100}$)	358.7	5547		1.302	1.221
26	R12MB	250	11.9	750	35.6	316.5	6174		1.218	1.158
27	R08HB	250	7.65	750	82.4	358.7	7115		1.118	1.014
28	R12HB	250	11.9	750	82.4	316.5	7977		1.135	1.045
29	S10D-2A	100.2	2.16	300	25.7	300	609		1.363	1.266
30	S40D-2A	400.0	9.05	1200	27.3	330	8326	Yamamoto *et al.* (2003) (10 tests)	1.051	0.980
31	S10D-4A	100.1	2.17	300	53.7	300	851		1.296	1.169
32	S10D-6A	100.1	2.18	300	61.0	300	911		1.279	1.149
33	S10A-2A	101.1	5.84	300	27.3 ($f_{cyl,100}$)	455	1293		1.086	1.058
34	S20A-2A	200.2	12.6	601	27.8	399	5266		1.179	1.147
35	S10A-4A	100.1	5.89	300	48.7	455	1375		1.042	1.000
36	S20A-4A	200.2	12.5	601	47.3	399	5714		1.159	1.112
37	S10B-2A-1	99.8	3.06	300	25.2	375	728		1.166	1.109
38	S20B-2A-2	99.8	3.07	300	25.2	375	726		1.160	1.104
39	CR4-A-2	148	4.38	444	25.4	262	1153	Sakino *et al.* (2004) (32 tests)	1.087	1.019
40	CR4-A-4-1	148	4.38	444	40.5	262	1414		1.088	1.001
41	CR4-A4-2	148	4.38	444	40.5	262	1402		1.078	0.992
42	CR4-A-8	148	4.38	444	77.0	262	2108		1.123	1.00
43	CR4-C-2	215	4.38	645	25.4	262	1777		0.96	0.887
44	CR4-C-4-1	215	4.38	645	41.1	262	2424		1.013	0.916
45	CR4-C4-2	215	4.38	645	41.1	262	2393		1.000	0.905
46	CR4-C-8	215	4.38	645	80.3	262	3837		1.022	0.904

Table 2 Continued

No. of specimens	Name of specimens	Dimensions B (mm)	t (mm)	L (mm)	Material properties f_c (MPa)	f_y (MPa)	Axial capacity N_{test}(kN)	Tested by	$M = N_{test}/N_{RA}$ AISC	Eurocode 4
47	CR6-A-2	144	6.36	432	25.4	618	2572	Sakino *et al.* (2004) (32 tests)	1.020	0.995
48	CR6-A-4-1	144	6.36	432	40.5	618	2808		1.027	0.991
49	CR6-A4-2	144	6.36	432	40.5	618	2765		1.012	0.976
50	CR6-A-8	144	6.36	432	77.0	618	3399		1.047	0.989
51	CR6-C-2	120	6.36	633	25.4	618	3920		1.048	1.032
52	CR6-C-4-1	120	6.36	633	40.5 ($f_{cyl,100}$)	618	4428		1.044	1.020
53	CR6-C4-2	120	6.36	633	40.5	618	4484		1.045	1.021
54	CR6-C-8	119	6.36	633	77.0	618	5758		1.057	1.017
55	CR8-C-2	175	6.47	525	25.4	835	4210		1.006	0.983
56	CR8-C-4-1	175	6.47	525	40.5	835	4493		0.996	0.964
57	CR8-C4-2	175	6.47	525	40.5	835	4542		1.007	0.974
58	CR8-C-8	175	6.47	525	77.0	835	5366		1.014	0.961
59	CR8-D-2	210	6.47	795	25.4	835	6546		1.233	1.135
60	CR8-D-4-1	211	6.47	795	41.1	835	7117		1.110	0.989
61	CR8-D-4-2	210	6.47	795	41.1	835	7172		1.173	1.068
62	CR8-D-8	211	6.47	795	80.3	835	8990		1.076	0.951
63	CR6-A-4-3	211	8.83	633	39.1	536	5898		1.175	1.128
64	CR6-A-9	211	8.83	633	91.1	536	7008		1.061	0.988
65	CR6-C-4-3	204	5.95	612	39.1	540	4026		1.081	1.024
66	CR6-C-9	204	5.95	612	91.1	540	5303		1.003	0.919
67	CR8-A-4-3	180	9.45	540	39.1	825	6803		1.107	1.081
68	CR8-A-9	180	9.45	540	91.1	825	7402		1.021	0.975
69	CR8-C-4-3	180	6.60	540	39.1	824	5028		1.079	1.044
70	CR8-C-9	180	6.60	540	91.1	824	5873		1.006	0.946
71	sczs1-1-1	120	3.8	360	27.3	330.1	882	Han and Tao (2001) (20 tests)	1.072	1.020
72	sczs1-1-2	120	3.8	360	31.2	330.1	882		1.025	0.970
73	sczs1-1-3	120	3.8	360	31.2	330.1	921		1.070	1.012
74	sczs1-1-4	120	3.8	360	49.3	330.1	1080		1.075	1.000
75	sczs1-1-5	120	3.8	360	52.5	330.1	1078		1.037	0.963
76	sczs1-2-1	140	3.8	420	16.0	330.1	941		1.077	1.037
77	sczs1-2-2	140	3.8	420	16.7	330.1	922		1.045	1.005
78	sczs1-2-3	140	3.8	420	54.6	330.1	1499		1.112	1.023
79	sczs1-2-4	140	3.8	420	54.6	330.1	1470		1.091	1.003
80	sczs2-1-1	120	5.9	360	30.0 ($f_{cu,150}$)	321.1	1176		1.056	1.016
81	sczs2-1-2	120	5.9	360	30.0	321.1	1117		1.003	0.965
82	sczs2-1-3	120	5.9	360	25.8	321.1	1196		1.116	1.079
83	sczs2-1-4	120	5.9	360	52.5	321.1	1460		1.134	1.072
84	sczs2-1-5	120	5.9	360	52.5	321.1	1372		1.066	1.007

Table 2 Continued

No. of specimens	Name of specimens	Dimensions B (mm)	Dimensions t (mm)	Dimensions L (mm)	Material properties f_c (MPa)	Material properties f_y (MPa)	Axial capacity N_{test}(kN)	Tested by	$M = N_{test}/N_{RA}$ AISC	$M = N_{test}/N_{RA}$ Eurocode 4
85	sczs2-2-1	140	5.9	420	16.3	321.1	1343	Han and Tao (2001) (20 tests)	1.121	1.091
86	sczs2-2-2	140	5.9	420	18.3	321.1	1293		1.059	1.029
87	sczs2-2-3	140	5.9	420	54.6	321.1	2009		1.226	1.149
88	sczs2-2-4	140	5.9	420	54.6	321.1	1906		1.163	1.090
89	sczs2-3-1	200	5.9	600	17.6	321.1	2058		1.086	1.045
90	sczs2-3-2	200	5.9	600	17.6	321.1	1960		1.034	0.995
91	S3	100.7	9.6	301	30.8	400	1550	Lam and Williams (2004) (10 tests)	1.004	0.988
92	S4	101	9.6	300	93.6	400	2000		1.081	1.036
93	S5	99.9	4.9	301	30.8	289	800		1.120	1.073
94	S6	99.8	4.9	300	93.6	289	900		0.834	0.766
95	S7	100.1	4.2	301	34.7 ($f_{cu,150}$)	333	700		0.947	0.903
96	S9	100	4.1	299	97.2	333	1130		1.016	0.929
97	S12	100	4.1	301	57.6	333	880		1.018	0.951
98	S14	101	9.6	302	57.6	400	1800		1.075	1.045
99	S16	99.7	4.7	301	58.2	289	1000		1.176	1.100
100	S18	99.9	4.1	301	98.9	333	1130		1.006	0.920
								Mean	1.094	1.028
								Standard deviation	0.082	0.077
								Maximum	1.363	1.266
								Minimum	0.834	0.766
								Maximum-Minimum	0.529	0.500

Table 3 Conversion relations between $f_{cyl,150}$ and $f_{cu,150}$ (Eurocode 2 2004)

$f_{cyl,150}$ (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90
$f_{cu,150}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105

corresponds to the stress at 0.2% offset). The steel tube and concrete are loaded simultaneously for all the 100 experimental specimens and the ultimate axial capacities of the square CFT stub columns obtained from experiments (N_{test}) correspond to the maximum (peak or limit points) of the axial load-shortening curves.

It should be noted that different test standards are used to define the compressive strength of concrete by the researchers, which are specified clearly in Table 2. The conversion relations between $f_{cyl,150}$ and $f_{cu,150}$ can be found in Eurocode 2 (2004) as shown in Table 3. The conversion relationship between $f_{cyl,150}$ and $f_{cyl,100}$ can be expressed as (Rashid *et al.* 2002)

$$f_{cyl,150} = 0.96 f_{cyl,100} \tag{8}$$

4. Statistical analysis of design code model error

In order to compare theoretical resistance models of the design codes with experimental results, a modeling uncertainty parameter (M) variable is introduced

$$M = \frac{N_{test}}{N_{RA}} \tag{9}$$

This has also been called the professional factor by Ellingwood and Galambos (1982). Samples of the modeling uncertainty parameter obtained for each design code provision, from the 100 available experimental results are also presented in Table 2 and are depicted in Fig. 2. In Fig. 2, specimens with a material strength beyond the limitations of the corresponding design codes (see Table 4) are depicted as the void circle, while the solid circle is denoting the specimens in the limitations of design codes.

Table 2 and Fig. 2 reveal the following:

(1) Generally, for the CFT specimens with material strength beyond the limitations of the design codes of Eurocode and AISC, the predictions by the corresponding design methods have almost the same trend as those in the limitations of design codes.

(2) The AISC code conservatively predicts a resistance nearly 9.4% lower than the mean of experimental results mainly due to the fact that the composite action between the steel tube and the concrete core was not considered. Because the beneficial concrete confinement by the steel tube is considered in the resistance model of Eurocode 4, it is able to predict mean column resistances.

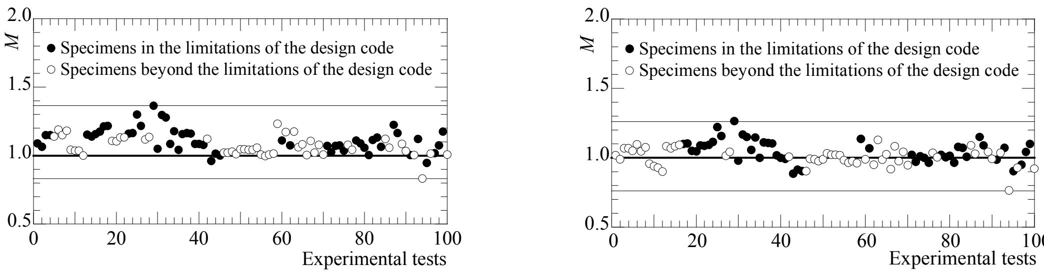


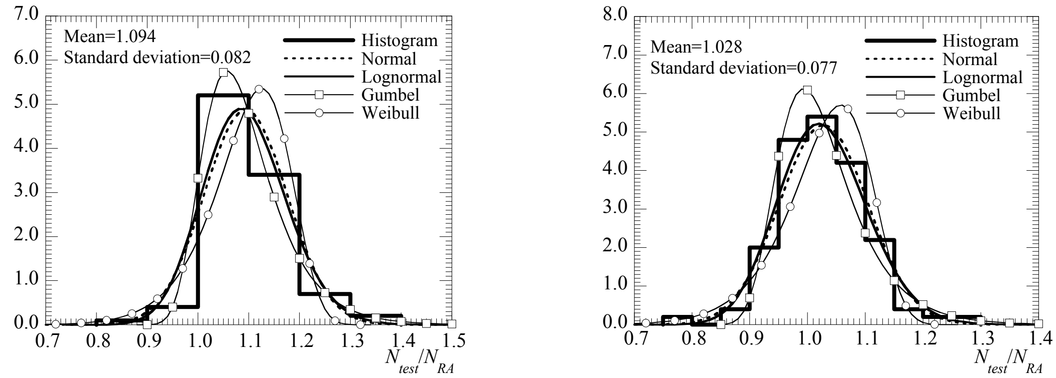
Fig. 2 Comparison of experimental results with predictions of AISC and Eurocode 4

Table 4 Limitations of applications of concerning the material's strength and width-to-thickness ratio of square steel tubes in AISC and Eurocode 4 for square CFT columns

	AISC (2005)	Eurocode 4 (2004)
Limitations of strength of steel tube (f_y, MPa)	$f_y \leq 525$	$235 \leq f_y \leq 460$
Ranges of compressive strength of normal weight concrete ($f_{cyl,150}$, MPa)	$21 \leq f_{cyl,150} \leq 70$	$20 \leq f_{cyl,150} \leq 50$
Limitations of width-to-thickness ratio of square steel tube (B/t)	$\leq 2.26 \cdot (E_s/f_y)^{0.5}$	$\leq 52 \cdot (235/f_y)^{0.5}$

The statistics moments including mean and standard deviation, and the histograms of the modeling uncertainty parameter of the two design code provisions for square CFT stub columns are depicted in Fig. 3. Four two-parameter distributions, i.e., normal, lognormal, Gumbel and Weibull distributions are used here to fit the statistical data. The PDFs of these four distributions, with the same mean value and standard deviation as the statistical data are also depicted in Fig. 3. As can be observed from Fig. 3, the normal and lognormal distributions generally give almost the same fitting results and fit the histogram much better than the Gumbel and Weibull distributions.

Results of the Chi-square tests for the modeling uncertainty parameter of AISC design code provision of the four distributions are listed in Table 5, in which the goodness-of-fit tests were obtained using the following equation (Ang and Tang 2006)



(a) Model error of AISC specification (b) Model error of Eurocode 4 specification

Fig. 3 Histogram and statistical moments of model error variables and fitting results

Table 5 Chi-square test results for N_{test}/N_{RA} (AISC)

Intervals	Frequency	Predicted frequency				Goodness of fit			
		Normal	Lognormal	Gumbel	Weibull	Normal	Lognormal	Gumbel	Weibull
< 0.90	1	0.90	0.51	0.01	2.36	0.01	0.47	98.01	0.78
0.9-1.0	4	11.68	11.74	8.69	10.25	5.05	5.10	2.53	3.81
1.0-1.1	52	40.33	42.15	51.29	34.92	3.38	2.30	0.01	8.35
1.1-1.2	34	37.28	35.45	29.87	45.70	0.29	0.06	0.57	3.00
1.2-1.3	7	9.21	9.19	7.93	6.76	0.53	0.52	0.11	0.01
> 1.3	2	0.60	0.96	2.21	0.01	3.27	1.13	0.02	396.01
Sum	100	100	100	100	100	12.52	9.58	101.25	411.96

Table 6 Probability characteristics of modeling uncertainty parameters

Model uncertainty parameters	Distribution	Mean	Standard deviation
AISC	Lognormal	1.094	0.082
Eurocode 4	Lognormal	1.028	0.077

$$T = \sum_{i=1}^{k} (O_i - E_i)^2 / E_i \tag{10}$$

where O_i and E_i are the observed and theoretical frequencies, respectively, k is the number of intervals used, and T is a measure of the respective goodness-of- fit. As can observed from Table 5, the goodness-of-fit tests verify that the lognormal distribution has the best fit with $T = 9.58$ among the four distributions. Similar results can be obtained for modeling uncertainty parameter of the Eurocode 4 design provision for square CFT stub columns. Modeling uncertainty parameters like mean, coefficient of variation (COV) and probability distribution, obtained for the two design code provisions are summarized in Table 6.

5. Reliability analysis

5.1 Resistance variables

Resistance random variables with significant uncertainty include the modeling uncertainty parameter and the material parameters such as the yield strength of steel and the compressive strength of concrete. Statistical moments and probability distributions of steel and concrete strength parameters taken from Ellingwood and Galambos (1982) and Bartlett and MacGregor (1996) are shown in Table 7. Three characteristic values of steel strength are considered in the analysis: f_{yk} = {200, 300, 400} MPa. Four characteristic concrete strengths were considered: $f_{cyl,150,k}$ = {30, 50, 70, 90} MPa. Other resistance parameters like the dimension of the column are treated as deterministic variables because of the small variability.

5.2 Load variables

In order to evaluate the reliability of CFT stub columns in a service condition, uncertainty in the loading is also taken into account. A combination of dead load and live load is considered. Nominal value of these actions, D_n and L_n, are determined from column resistance, using the partial load factors recommended in design code provisions of AISC and Eurocode 4

$$N_{RD} = \gamma_D D_n + \gamma_L L_n \tag{11}$$

where γ_D and γ_L are the load factors for dead load and live load, respectively. The values of γ_D and γ_L in the code provisions of AISC and Eurocode 4 are shown in Table 1.

Eq. (11) is solved for the nominal load values D_n and L_n for a fixed load ratio L_n/D_n. Seven load ratios are considered in this study: L_n/D_n = {0.0, 0.25, 0.5, 1.0, 2.0, 3.0, 4.0, 5.0}. Statistical moments and probability distributions of these variables taken from Ellingwood and Galambos (1982) are also shown in Table 7.

5.3 Range of problem parameters considered

In order for the reliability analysis to reflect the range of design conditions covered by the codes in study, a range of design parameters has to be considered. As described earlier, in this study 3 values of the steel yield strength, 4 values of concrete strength and 7 load ratios are

Table 7 Random variable data for reliability analysis

Random variable	Distribution	Mean	COV	Reference
Steel yield strength, x_1	Lognormal	$1.08f_{yk}$	0.05	Ellingwood and Galambos (1982)
Concrete compressive strength, x_2	Normal	$1.08f_{cyl,150,k}$	0.15	Bartlett and MacGregor (1996)
Model uncertainty parameters, x_3	Lognormal			Following Table 6
Dead load, x_4	Normal	$1.05D_n$	0.10	Ellingwood and Galambos (1982)
Live load, x_5	Gumbel	$1.00L_n$	0.25	Ellingwood and Galambos (1982)

considered. Moreover, the study considers 2 tube thickness: t = {10; 20} mm and 2 tube width: B = {400; 600} mm. In total, 336 column configurations are covered in the reliability analysis.

5.4 Limit state function for the reliability analysis

The limit state function G(X), for the reliability analysis can be given by

$$G(\mathrm{X}) = N_{RA}(x_1, x_2) \cdot x_3 - x_4 - x_5 \tag{12}$$

where N_{RA} is the code resistance model; x_1 is the steel yield strength; x_2 is the concrete compressive strength; x_3 is the model uncertainty parameters; x_4 is the dead load; and x_5 is the live load. Reliability indices of the limit state function Eq. (12) are evaluated using the First-Order Reliability Method (FORM) (Nowak and Collins 2000).

5.5 Reliability analysis results

Reliability analysis results are presented in this section. Fig. 4 shows the reliability index (β) results for the two design codes considered as function of load ratios. Two sets of results are shown in this figure. The continuous lines represent actual column reliability, since these results include the modeling uncertainty parameter. The dashed lines are obtained when modeling uncertainty parameter is not considered, and are shown only to illustrate the impact of modeling uncertainty parameter in reliability results. Each result set corresponds to two lines, showing the range of reliability indexes obtained. In other words, the upper and lower bounds of β amongst all analyzed columns are shown in Fig. 4. It can be observed that:

(1) The modeling uncertainty parameter has significant impact in reliability. Generally, for small load ratios, modeling uncertainty parameter reduces reliability indices for the two codes. This is because for small load ratios, the most important contribution to failure probabilities comes from model error. Although the resistance models are conservative with the modeling uncertainty parameter mean greater than one, the modeling uncertainty parameter variance impacts adversely on reliability indices.
(2) For the AISC codes, and for large load ratios, a consideration of modeling uncertainty parameter increases the reliability indices. This happens due to conservativeness of the code in the design of square CFT stub columns.
(3) For larger load ratios (L_n/D_n > 2.0), the modeling uncertainty parameter has virtually no impact in Eurocode 4 results. The reason may lie in that although the resistance model is slightly conservative, with the modeling uncertainty parameter mean value of 1.028,

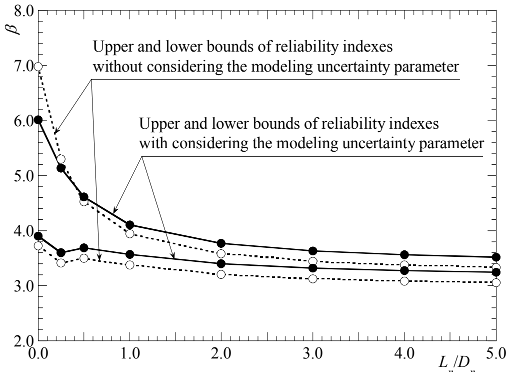
modeling uncertainty parameter variance still impacts adversely on reliability indices. The net result is no impact of modeling uncertainty parameter in reliability results.

Summary of reliability index results including the minimum, maximum, and the range of reliability indices obtained for the two design codes is shown in Table 8. Clearly, such limits are not absolute: they reflect the range of square CFT stub column configurations considered in the reliability analysis, which are believed to reflect the universe of practical column configuration covered by the design codes.

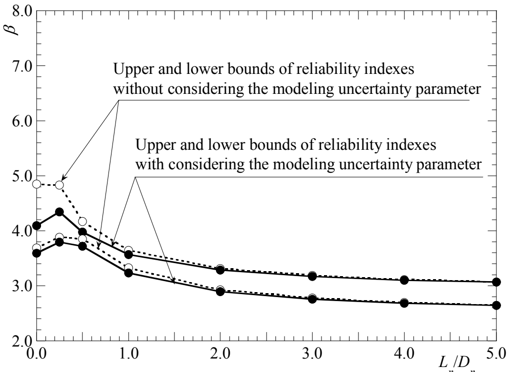
Two criteria can be used to evaluate design code provisions in terms of reliability: structural design codes should provide sufficient and uniform reliability over the range of designs covered by the code (Ellingwood and Galambos 1982).

The sufficiency criterion requires a definition of an acceptable level of safety. Reliability indices used in code calibration work should be useful in this purpose. According to the AISC code, the target reliability index is equal to 2.6 for the *Dead+Live* load combination. Fig. 4(a) and Table 8 show that the AISC code meets the target and is conservative with a minimum reliability index of 3.25. The authors ignore target β values used in the calibration of Eurocode 4, but its Annex C recommends a minimum β_T = 3.8 for 50 years and consequence class 2 (residential and office buildings). This level of reliability is not achieved by the Eurocode 4, as shown in Fig. 4b and Table 8. Our results show reliability indexes as low as β = 2.64.

The two design codes studied seem to provide reasonably uniform reliability indices, as shown in Fig. 4 and Table 8. Overall, the Eurocode 4 is better at achieving this goal with a reliability index range of 1.701 compared with the range of 2.111 achieved by AISC. For load ratios L_n/D_n < 2.0, the uniformness of reliability indices for the AISC code is no better because of the quality of



(a) AISC



(b) Eurocode 4

Fig. 4 Reliability indexes of code-designed columns as function of load ratio

Table 8 Summary for reliability index results

Design code	$\beta_{\min}$	$\beta_{\max}$	$\beta_{\max} - \beta_{\min}$
AISC	3.247	6.013	2.111
Eurocode 4	2.642	4.343	1.701

the resistance model, in which composite action between the steel tube and the concrete core was not considered. While for load ratios $L_n/D_n \geq 2.0$, the uniformness of reliability indices for the AISC code is better. The interpretation of the results involves the following point. In terms of load combinations, the ratio of dead to live load factors of AISC (1.2/1.6 = 0.75) seems to better balanced, at least in light of the calibrations performed by Ellingwood and Galambos (1982). For the Eurocode 4, the ratio is greater (1.35/1.5 = 0.9), hence less likely to reflect differences in the uncertainties of Live and Dead Loads, resulting in a larger scatter of reliabilities.

6. Conclusions

This paper presented a probabilistic investigation of design code provisions such as AISC and Eurocode 4 for square CFT stub columns using experimental results of 100 axially loaded square CFT stub columns published in the literature. It is found that:

(1) Generally, for the CFT specimens with material strength beyond the limitations of the design codes of Eurocode and AISC, the predictions have almost the same trend as those in the limitations of design codes. Resistance model of Eurocode 4 provide detailed allowances for concrete confinement by the steel tube. As a result, it is able to predict mean column resistances. Because the beneficial confining effect has not been taken into consideration in the design code of AISC, it gives a sectional capacity about 9.4% lower than the experimental results.

(2) Modeling uncertainty parameter is quite relevant for column safety, especially for small load ratios. In general, consideration of modeling uncertainty parameter resulted in increased reliability of stub columns designed according to AISC code, due to conservative column resistances predicted by the code. While for design codes of Eurocode 4, consideration of modeling uncertainty parameter produces a reduction of reliability indices.

(3) Modeling uncertainty parameter can be described by random variables of lognormal distribution.

(4) The AISC design code provided slightly conservative results of square CFT stub columns with reliability indices larger than 3.25 and the uniformness of reliability indices are no better because of the quality of the resistance model.

(5) The uniformness of reliability indices for the Eurocode 4 was better than that of AISC, but the reliability indices of columns designed following the Eurocode 4 were found to be quite below the target reliability level of Eurocode 4.

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Notation

A_c	cross-section area of the concrete
A_s	cross-section area of the steel tube
B	width of square steel tube
D_n	nominal value of dead load
E_c	modulus of elasticity of concrete
E_s	modulus of elasticity of steel
E_i	theoretical frequencies
$(EI)_{eff}$	effective stiffness of composite section
f_c	concrete compressive strength
$f_{cyl,100}$	concrete compressive strength obtained from cylinder tests of 100X200 mm specimen
$f_{cyl,150}$	concrete compressive strength from cylinder test of 150X300 mm specimen
$f_{cyl,150,k}$	characteristic concrete strength
$f_{cu,150}$	concrete compressive strength from 150 mm cube tests
f_y	yield stress of steel tube
f_{yk}	characteristic value of steel strength
I_s	moment of inertia of steel tube
I_c	moment of inertia of concrete core
k	number of intervals used
K_A	effective length factor
L	lengths of the columns
L_A	laterally unbraced length of the column
L/B	length-to-width ratio
L_n	nominal value of live load
M	modeling uncertainty parameter
N_{RA}	actual resistance (without safety margins and with mean material values)

N_{RD}	design resistance of a CFT column
N_{test}	ultimate axial capacities of the square CFT stub columns obtained from experiments
O_i	observed frequencies
$P_{0,AISC}$	capacity of the cross section (zero length strength)
P_e	elastic buckling load
P_n	nominal resistance
T	measure of goodness-of- fit
t	wall thickness of the steel tube
B/t	width-to-thickness ratio
β	reliability index
$\beta_{\min}$	minimum of reliability indices
$\beta_{\max}$	maximum of reliability indices
$\beta_{\max}$- $\beta_{\min}$	range of reliability indices
β_T	target reliability index
ϕ_c	partial factor applied to column resistance
γ_a	partial resistance factors for steel
γ_c	partial resistance factors for concrete
γ_D	load factors for dead load
γ_L	load factors for live load