



## BUCKLING OF SLENDER COMPOSITE CONCRETE-FILLED STEEL COLUMNS

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**Abstract.** The paper presents the analysis of the experimental data of 1817 on concrete-filled steel tubes – CFSTs. These results are compared with the predicted results of the load-bearing capacity of calculations of slender elements according to the methods suggested by Eurocode 4. The following types of tested CFSTs were analysed: circular and rectangular hollow section stub and long columns fully filled with concrete, which were with or without applied moments at the ends of specimen. During the results obtained in the result of the tests on the load bearing capacity for circular concrete-filled steel tubular columns correspond with the calculated values based on methods presented by Eurocode 4. The experimental values of load bearing capacity for members of concrete-filled rectangular hollow sections agree very well with the theoretical values where the concrete cylinder strength is below 75 N/mm<sup>2</sup>. The analysis demonstrated that preloading of concrete-filled hollow section members does not influence the load bearing capacity.

This paper also presents the examination of stress state distribution for concrete-filled hollow section members, influence of concrete preloading and of longitudinal stress strain curves.

**Keywords:** concrete-filled steel columns, buckling, slenderness, load-bearing capacity, Eurocode 4, analysis, comparison, test results.

### 1. Introduction

Steel structural hollow sections are the most efficient of all the structural sections in resisting compression load. And filling these sections with plain concrete significantly increases load-bearing capacity. CFST columns have a number of advantages as follows: a) it combines tension properties of steel and compression properties of concrete and provides the hollow steel sections with greater strength and stiffness, b) the confinement of concrete by steel enhances failure strength of concrete, c) column size may be reduced more than necessary for pure steel or RC column and provide greater floor area for use, d) the steel tube provides the permanent formwork for concrete, e) steel tube column can be erected rapidly for a number of storey heights, allowing to add floors before filling tubes with concrete, f) good seismic resistance because of good ductility and high energy absorbing properties, etc. (Kvedaras *et al.* 2006, 2009; Partaukas and Bareisis 2009; Benzaid *et al.* 2008; Soundararajan *et al.* 2008; Kuranovas *et al.* 2007; Goode 2007; Kuranovas and Kvedaras 2007; Kudzys *et al.* 2006; Kuranovas 2006; Eurocode 4 2005; Han *et al.* 2004; Baochung and Hiroshi 2003; Gopal and Manoharan 2003; Han and Yao 2002, 2003; Han and Yang 2003; Chung *et al.* 2001; Han 2000; DL/T5085 1999; Zhang and Zhong 1999; Zhong 1999).

The main disadvantage is the degradation of steel properties when exposed to fire, but these elements may be protected by spraying fire retardants to outer surface of

the steel tube or by using plasterboard and rock-wool insulation. Furthermore, load-bearing capacity under fire may be improved by using internal reinforcement bars.

Different approaches and design philosophies were adopted in different design codes of different countries. But still in present time it is a lack of information for designers: investigations, test results, FEM and structural analyses are necessary to derive more precise and evaluating safer methods for such type elements. That's why C. D. Goode started collecting data for database, analysed it and proposed new suggestions how to predict more precisely load-bearing capacity of various type CFST elements under various type loading.

### 2. Codes and test data

Different limitations on the compressive strength of concrete, steel yield strength, diameter-to-thickness ratio; steel ration and confining coefficient are prescribed in different codes (Xinbo *et al.* 2006). These limitations are compared in Table 1 (Kuranovas *et al.* 2009); where  $\xi = 1,5A_a f_{ay} / A_c f_{ck}$ ,  $f_{ck}$  is the 150 mm cube compressive strength of concrete;  $f_{ay}$  – is the yield strength of steel tube,  $A_a$ ,  $A_c$  are areas of steel tube and concrete core respectively,  $a_a$  is steel ratio; and  $E$  is the elasticity modulus of steel tube.

**Table 1.** Comparison of the limitations in different codes

Item	CHN-JCJ 01-89	CHN CECS	CHN-DL/T 5085
$f_{ck}$	30~50	30~80	30~80
$f_{ay}$	235~345	235~420	235~390
$D/t_a$	~	$20 \sim 90\sqrt{235/f_{ay}}$	20~100
$a_a$	0,04~0,16	–	0,04~0,20
$\xi$	–	0,03~0,3	–
Item	AISC-LRFD(99)	Eurocode 4	JAN-AIJ(97)
$f_{ck}$	26~65	25~60	
$f_{ay}$	$\leq 415$	235~355	
$D/t_a$	$\leq \sqrt{8E/f_{ay}}$	$\leq 90\sqrt{235/f_{ay}}$	$\leq 35280/f_{ay}$
$a_a$	$\geq 0,04$	–	–
$\xi$	–	–	–

The limitations and conditions for composite columns and composite compression members according to EC4 are: a) the steel grade should be S235 to S460, b) concrete of strength classes C20/25 to C50/60, c) local buckling can be neglected if  $t > h/(52\sqrt{235/f_y})$  and  $t > D/(90\sqrt{235/f_y})$  for columns of circular and rectangular cross section respectively.

Overall buckling is allowed for in the EC4 by introducing a buckling factor  $\chi$  related to the relative slenderness  $\bar{\lambda}$  by the European buckling curve:

$$\chi = 1 / \left( \varphi + \sqrt{\varphi^2 - \bar{\lambda}^2} \right),$$

where:  $\varphi = 0.5 \cdot (1 + \alpha(\varphi - 0.2) + \bar{\lambda}^2)$  and

$$\bar{\lambda} = \sqrt{N_{Pl,Rd} / N_{cr}},$$

$\alpha = 0.21$  – for any added reinforcement being  $\leq 3\%$  and  $\alpha = 0.34$  – for  $3\% < \rho_s \leq 6\%$ ;

$$\rho_s = 100 \times A_a / A_c;$$

$N_{Pl,Rd} = A_a f_{y,d} + A_c f_{cd} + A_s f_{fsd}$  – plastic compression resistance;

$N_{cr} = \pi^2 (EI)_{eff} / L^2$  – the elastic critical load;

$$(EI)_{eff} = E_a I_a + E_s I_s + 0.6 E_{cm} I_c.$$

When columns are loaded by end moment two methods of analysis can be applied:

1. Simplified method, where the second-order effects are allowed for by multiplying the first-order applied moments by a factor ‘k’;

2. More precisely method, where the second order effect is analysed.

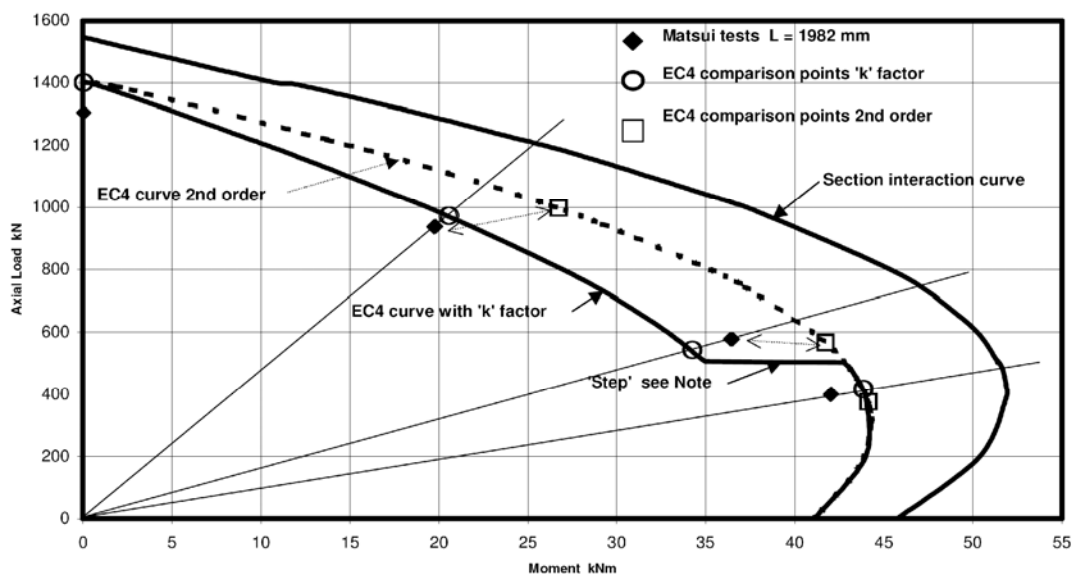
For columns with equal end moments ‘k’ is given by:

$$k = 1,10 / (1 - (N_u / N_{cr,eff})) \geq 1,00.$$

In the simplified method the calculated moment resistance has been divided by the ‘k’ factor to compare with the test result (rather than factoring the test result by ‘k’) and the failure load predicted by the code is compared with the test result at the same axial load/moment ratio as was used in the test. Fig. 1 illustrates this where the Eurocode 4 curve allows for slenderness ( $\chi$ ), the steel grade factor  $\alpha_M$ , and the ‘k’ factor. Factor  $\alpha_M$  has been taken as 0.9, when steel yield strength in the test was less than 420 N/mm<sup>2</sup> and 0.8, when greater than this.

Thus in the EC 4 curve  $M_{ECA} = \chi \times \alpha_M \times M / k$ . Eurocode 4 states that second-order effects need not be considered and that when the applied load divided by the elastic critical load is less than 0.1, ‘k’ may be taken as unity.

This results in a ‘step’ in the interaction diagram as when  $N / N_{cr,eff} < 0.1$  then  $k = 1$  but when  $N / N_{cr,eff} = 0.1$  then  $k = 1.22$ . Test results by Matsui *et al.* are also

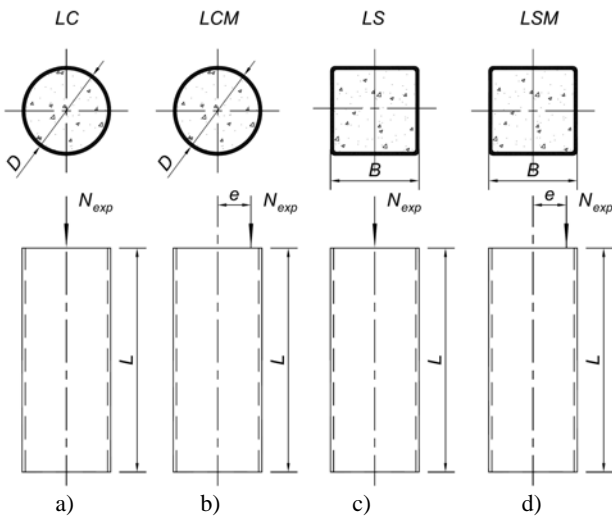


**Fig. 1.** Typical axial load–moment interaction curve. Note: The ‘step’ in the Eurocode 4 curve is because of that ‘second-order effects need not be considered when  $N_{Ed} / N_{cr,eff} < 0.1$  the factor ‘ $k$ ’ is unity but for  $N_{Ed} / N_{cr,eff} = 0.1$  ‘ $k$ ’ = 1.22

shown (Goode 2009). When the ‘second order’ method is used the lateral deflection of the column at mid-height caused by the end moment was added to the end eccentricity of the load to give the second-order moment and it is the load at this moment on the second-order curve which is compared to the failure load of the column.

In this paper data of results were used to form website database (<http://web.ukonline.co.uk/asccs2>) (Eurocode 4 2005), which collects together information for 1819 tests of CFSTs and compares the test results with EC4; some typical graphs are also included into this paper.

The data collected in the database is subdivided into “long” ( $L/D(B) > 4$ ) columns of “circular” and “square” cross-section (Fig. 2).



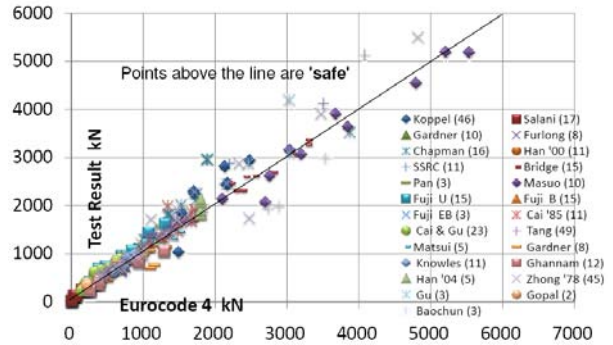
**Fig. 2.** Long CFST slenderness columns  $L/D(B) > 4$ : a) circular CFST with no moment, b) circular with moment, c) square CFST with no moment, d) square CFST with moment

The information required and reported for each test is: outer diameter ( $D$ ) if circular cross-section, or breadth ( $B$ ) and depth ( $H$ ) if rectangular one; the thickness ( $t_a$ ) of the steel tube; the steel properties ( $f_{ay}$ ) and, for slender columns, modulus of elasticity ( $E_a$ ); the concrete properties (concrete yield strength ( $f_{cyl}$ ), ( $f_{ck}$  in Goode 2008) and, for long columns, its secant modulus of elasticity ( $E_c$  to  $0,4f_{ck}$ )); the length ( $L$ ) of the column; the maximum load achieved by the column in test ( $N_u =$  Test failure load). For columns with an end moment the initial eccentricity of load at the top ( $e_t$ ) and bottom ( $e_b$ ) is required.

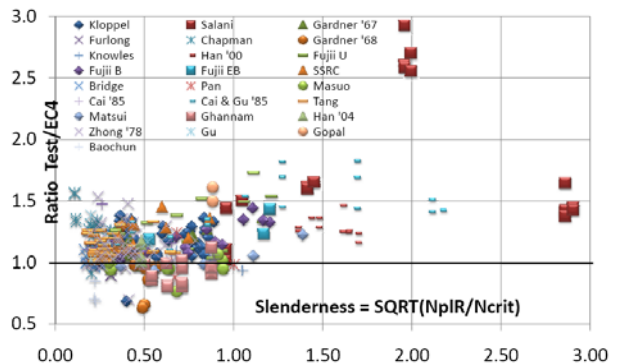
Dispersions of EC4 vs. test and ratio test/EC4 vs. concrete strength for LS and LSM are presented in Figs 7–8 and 9–10 respectively.

Summary results of LC, LCM, LS and LSM are presented in Table 2 (Yamamoto *et al.* 2000), where it can be seen that the average values of test/EC 4 for each type of column are higher than Eurocode 4 predicts.

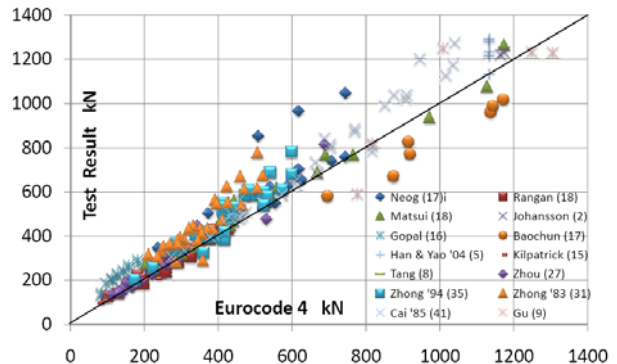
Dispersions EC4 vs. test and ratio test/EC4 vs. concrete strength for slenderness elements are shown in Figs 3–10.



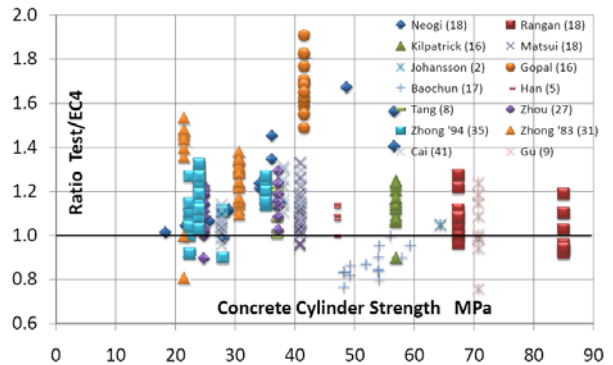
**Fig. 3.** Tests results of LC concrete-filled steel columns compared with calculations according to (Goode 2008)



**Fig. 4.** Ratio test/EC4 vs. slenderness of LC concrete-filled steel columns



**Fig. 5.** Tests results of LCM concrete-filled columns compared with calculations according to (Goode 2008)



**Fig. 6.** Ratio Test/EC4 vs. concrete cylinder strength of LCM concrete-filled steel columns

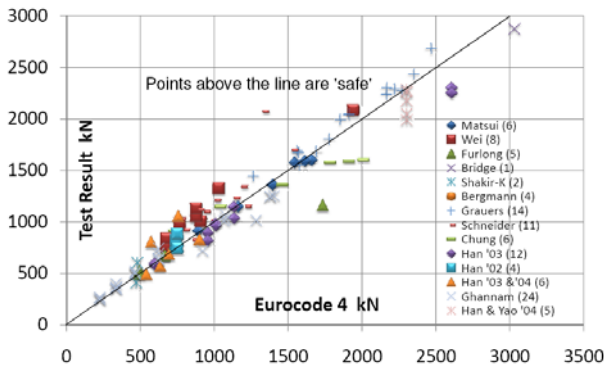


Fig. 7. Tests results of LS concrete-filled steel columns compared with calculations according to (Goode 2008)

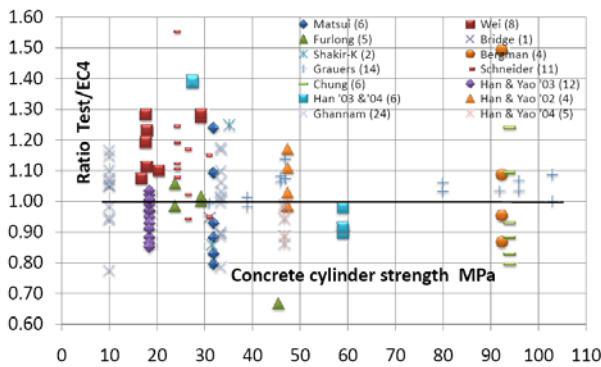


Fig. 8. Ratio test/EC4 vs. concrete strength for LS concrete-filled steel columns

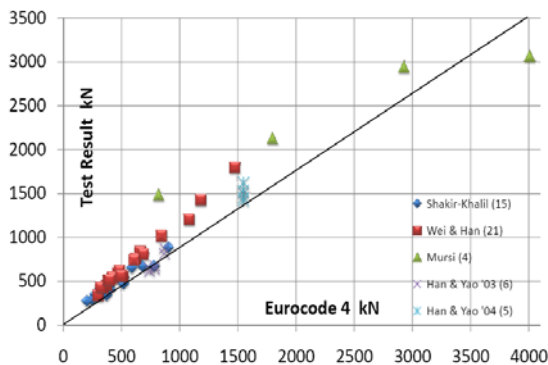


Fig. 9. Tests results of LSM concrete-filled steel columns compared with calculations according to (Goode 2008)

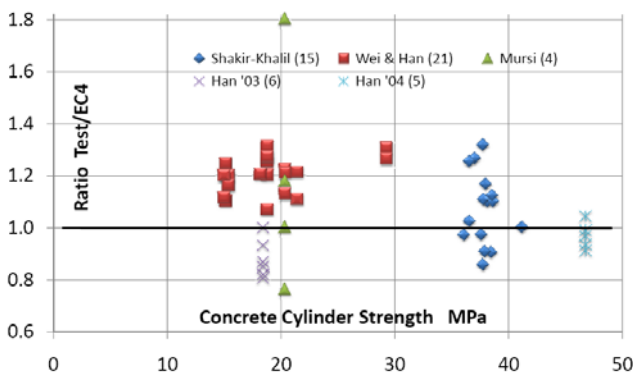


Fig. 10. Ratio test/EC4 vs. concrete cylinder strength of LSM concrete-filled steel columns

The ‘failed tests’ column of Table 2 shows the percentage of all tests failed before reaching the resistance calculated by EC4 methods.

The last column shows this percentage for the columns, which met all conditions indicated in Eurocode 4. Table 3 (Yamamoto *et al.* 2000) shows a comparison between average test/EC 4 ratio for different concrete strengths and also the local buckling criteria for the different types of column. For LC type of concrete-filled steel column, there occurred few ‘unsafe’ results, when the concrete cylinder strength was outside 20 to 50 N/mm<sup>2</sup> cylinder strength permitted by Eurocode 4 than when within this range. For the long square concrete-filled columns, a decrease in the test/EC4 ratio may be observed after high strength concrete has been used for the LCM type.

For circular section columns with an applied end moment, the simplified ‘*k*’ factor method and the ‘second order’ analysis showed very similar results; see Fig. 11 (Yamamoto *et al.* 2000). However, for rectangular columns the second order analysis predicted, in general, lower (safer) failure loads than the ‘*k*’ factor method and often much lower than achieved during the tests. Note that about 17% of all the circular and rectangular tests failed below their predicted failure load. The average test/EC 4 ratio for the 1027 tests analysed in this paper was 1.14 demonstrating standard deviation of 0.114.

### 3. Analysis of other type structures and loadings

#### Hollow Sections

The 76 hollow circular section and the 24 hollow square, 8-sided or 16-sided section columns without bending behaved in a similar way to the solid sections when allowance was made for the hole. The average test/EC 4 ratios were 1.22 for the circular sections and 1.16 for the other sections.

#### Preload and Sustained Load

Pre-load (up to 60% of the capacity of the steel) on the steel tube before filling with concrete seems to have no effect on the strength; the average test/EC 4 for the 23 circular columns (11 short and 12 long) being 1.15 (Standard deviation 0.123) and for the 19 rectangular columns (10 short and 9 long) being 1.03 (Standard deviation 0.099). The eight tests, which sustained an average load of between 53% and 63% of their capacity for 120 or 180 days before being loaded to failure, carried a slightly higher load before failing (average test/EC 4 = 1.25) than their six comparison tests without sustained load (average test/EC 4 = 1.08).

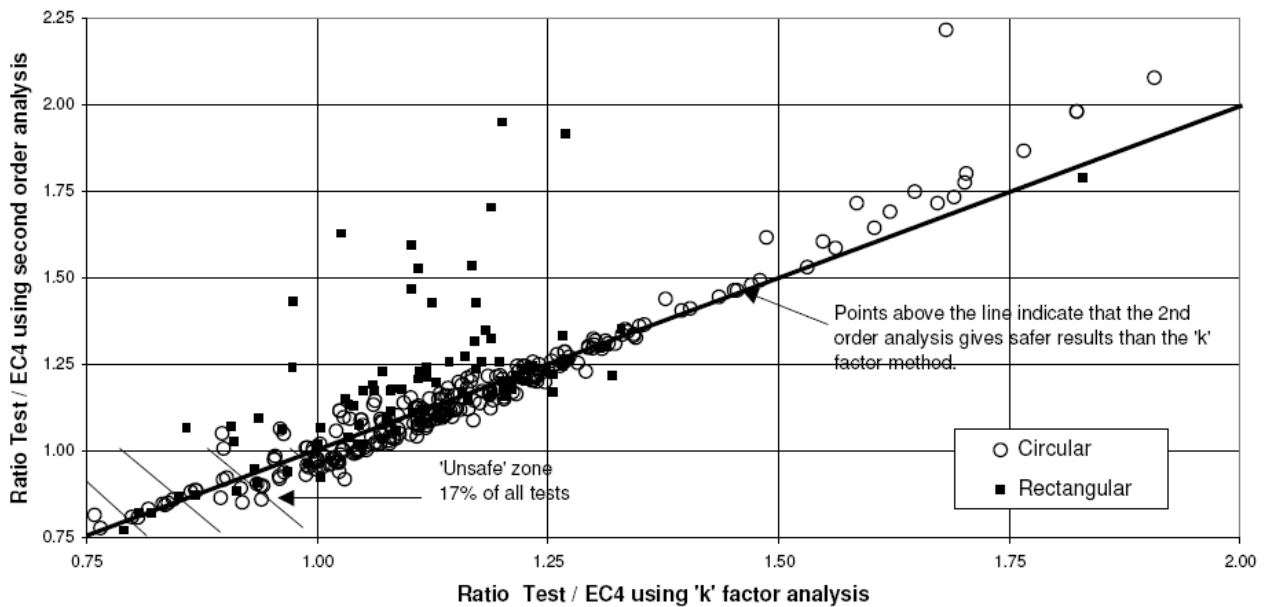
Yamamoto (2000) and Zhang *et al.*(2007) tested short circular concrete specimens with steel encasing the concrete and the axial load applied only to the concrete and not to the steel. Their results show that stub columns so loaded were able to sustain a higher load (19 tests test/EC 4 = 1.13) than similar sections where the load was applied uniformly over both steel and concrete (15 tests test/EC 4 = 1.03).

**Table 2.** Summary of results for each type of column

Type of column	NO. OF Tests	AVE Test/EC4	ST. DEV. of Test/EC4	% where Test/EC4 < 1		
				Failed tests	Tests which satisfy EC4	
					% < 1	% < 1
LC	369	1.17	0.148	16%	306	11%
LCM 'k' factor method	254	1.15	0.111	16%	198	10%
	2 <sup>nd</sup> order analysis	254	1.15	0.119	22%	198
LS	212	1.06	0.097	35%	76	30%
LSM 'k' factor method	96	1.11	0.098	18%	26	19%
	2 <sup>nd</sup> order analysis	96	1.20	0.148	11%	26
<b>Totals</b>	<b>1027</b>	<b>1.14</b>	<b>0.120</b>	<b>20%</b>	<b>830</b>	<b>15%</b>

**Table 3.** Comparison of Test/EC4 for different concrete strengths and for local buckling criteria

Type of column	$f_{cyl} \leq 50 \text{ N/mm}^2$		$f_{cyl} > 60 \text{ N/mm}^2$		$f_{cyl} > 75 \text{ N/mm}^2$		Local buckling criteria satisfied			Local buckling 'likely'		
	Ave	% < 1	Ave	% < 1	Ave	% < 1	No	Ave	% < 1	No	Ave	% < 1
LC	1.19	14	1.08	29	1.00	63	334	1.19	14	35	1.06	37
LR	1.05	40	1.09	26	1.04	35	130	1.07	32	82	1.05	40
LCM	1.18	10	1.05	34	1.01	56	255	1.15	16	none		
LRM	1.09	31	1.10	11	0.98	75	60	1.14	13	65	1.03	42



**Fig. 11.** Columns with moment; comparison of 'k' factor analysis with 2nd order analysis



### Biaxial Bending

Only 11 tests on rectangular columns with biaxial bending are reported and these all failed at much higher loads than predicted by Eurocode 4, average test/EC 4 was 1.52. The Code uses a straight line interaction for the bending resistance between the two axes with an additional safety factor  $\alpha_M$ , (with  $\alpha_M$  as 0.9 for steel grades S235 to S355 and 0.8 for steel grades S420 and S460). Using an elliptical interaction between the moments about the two axes and omitting this additional safety factor, i.e.  $\alpha_M = 1$ , gives much closer agreement with the test failure load, an average test/prediction of 1.20 for these 11 tests (Goode 2008, 2009).

### 4. Summary and future perspectives

Eurocode 4 predicts safe methods of strength evaluation for slender LC and LCM concrete-filled columns and could be safely used for concrete with cylinder strength up to 100 N/mm<sup>2</sup>. And C.D. Goode's opinion that for circular section columns the Code limitation on concrete cylinder strength could be safely extended to 75 N/mm<sup>2</sup>.

For rectangular section CFST columns Eurocode 4 should be used with caution, when the concrete cylinder strength is greater than 75 N/mm<sup>2</sup> as the failure load in the majority of tests when  $f_{cyl} > 75$  N/mm<sup>2</sup> was less than that predicted by the EC4 approach (Note: EC4 limits the concrete strength to 50 N/mm<sup>2</sup>).

C.D. Goode states that the concrete strength limitation for rectangular section columns could be safely extended to 60 N/mm<sup>2</sup>. When higher strength concrete is used, its cylinder strength should be factored by 0.85, equivalent to assumption that no enhancement of concrete strength should be experienced due to containment.

Sections, both circular and rectangular, which have a wall thickness thinner than permitted (Goode 2008) by the local buckling could be used if a factor of 0.75 was applied to the resistance predicted by Eurocode 4.

The simplified 'k' factor method and second order analysis of Eurocode 4 showed similar results.

Further investigations, tests, FEM and structural analyses are required.

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## LIAUNŲ BETONŠERDŽIŲ PLIENINIŲ VAMZDINIŲ KOLONŲ KLUPUMAS

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### S a n t r a u k a

Straipsnyje aptariami 1817 betonšerdžių plieninių vamzdinių strypų eksperimentiniai duomenys. Šie duomenys lyginami su rezultatais, gautais remiantis Eurocode 4 pateiktais kompozitinių elementų laikomosios galios nustatymo metodais. Analizuojami tokie betonšerdžių plieninių strypų bandinių tipai: pilnavidurės trumpos arba liaunos apskritojo ir stačiakampio skerspjūvio vamzdinės betonšerdės plieninės kolonos su jų galuose veikiančiu lenkiamuoju momentu arba be jo. Apskritojo skerspjūvio betonšerdžių kolonų bandymų metu gautieji laikomosios galios rezultatai atitinka remiantis Eurocode 4 pateiktais metodais apskaičiuotąsias jų reikšmes. Stačiakampio skerspjūvio betonšerdžių elementų laikomosios galios bandymais rastosios reikšmės labai gerai atitinka teorines reikšmes, kai šerdies betono ritininis stipris nesiekia  $75 \text{ N/mm}^2$ . Analizuojant nustatyta, kad išankstinis betonšerdžių elementų apkrovimas neturi beveik jokio poveikio elementų laikomajai galiai. Šiame straipsnyje taip pat nagrinėjamas betonšerdžių elementų įtempių būvių pasiskirstymas, betono apspaudimo poveikis bei išilginių deformacijų ir įtempių kreivės.

**Reikšminiai žodžiai:** kompozitinės konstrukcijos, betonšerdės plieninės vamzdinės kolonos, Eurocode 4, skaičiavimas, lyginimas, liaunumas, klupumas, laikomoji galia, bandymo rezultatai.

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