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LOAD-BEARING CAPACITY OF CONCRETE-FILLED STEEL COLUMNS

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Abstract. This paper represents the analysis of 1303 specimens of CFST experimental data. Test results are compared with EC4 provided method for determining the load-bearing capacity of these composite elements. Several types of CFSTs were tested: both circular and rectangular cross-sections with solid and hollow concrete core with axial load applied without and with moment, with sustained load and preloading. For circular cross-section columns there is a good agreement between the test failure load and the EC4 calculation for both short and long columns with and without moment. For rectangular cross-section columns the agreement is good except when the concrete cylinder strength was greater than 75 MPa, when many tests failed below the strength predicted by EC4. Preloading the steel tube before filling with concrete seems to have no effect on the strength. This paper also presents the stress distribution, confinement distribution and complete average longitudinal stress-strain curves for concrete-filled steel tubular elements. Based on the definition of the "Unified Theory", the CFST is looked upon as an entity of a new composite material. In this paper, the research achievement of the strength and stability for centrifugal-hollow and solid concrete filled steel tube are introduced. These behaviours relate to the hollowness ratio and the confining indexes of corresponding solid CFST. If the hollow ratio equals to 0,4–0,5 and over, the N-ε relationship exists in steady descending stage. The critical stress of CFST elements stability is determined as an eccentric member with the initial eccentricity by use of finite element method.

Keywords: composite structures, concrete-filled steel tubes, Eurocode 4, comparison, analysis load-bearing capacity, hollow concrete-filled steel tubes, behaviour, stress state, Poisson's ratio, elasticity modulus.

1. Introduction

Concrete-filled steel tubular (CFST) structures is a type of the composite steel-concrete structures used presently in civil engineering and consists of steel tube and concrete core inside it. The steel tube acts as a permanent formwork and can be of various cross-sections: circular, rectangular, square and multi-side.

According to the form of concrete core, CFST members can be divided into 2 types: with solid and hollow concrete core. Elements with solid core are formed by placing plain concrete into the steel tube with compaction of it by vibrating. The hollow CFST is produced by spinning method. The point of production by spinning is that during this process in the uniformly distributed plastic wet concrete centrifugation pressure appears, as the result of distances between aggregates and other solid particles; and wet concrete diminishes and "excess" water weakly bonded with other particles is pressed out of concrete substance. Increasing the concrete density helps to retain the achieved form.

Steel structural hollow sections are the most efficient of all the structural sections in resisting compression load. By filling these sections with concrete either a significant increase in load bearing capacity is achieved or the column size can be reduced. CFST columns have many advantages over reinforced concrete columns.

2. Overview of existing design codes for CFSTs

Different design regulations were produced for various cross-sections of CFST structures. Different approaches and design philosophies have been adopted in different design codes (Xinbo *et al.* 2006). In China, there are circular CFST structure design regulation, square structure design regulation, rectangular structure design regulation, and circular hollow CFST structure design regulation. In these regulations, the design methods are different. In China and Japan, the standard for designing the composite columns is based on a simple method of superposition that uses the allowable stresses of the materials or the working stress method. ACI-318 adopts the traditional reinforced concrete approach. AS 3600-1994 also uses the concept of reinforced concrete design. The AISC-LRFD is based on the concept of structural steel. The Eurocode 4, being a dedicated code

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for composite construction, combines the design approach of both structural steelwork and reinforced concrete columns.

Different limitations on the compressive strength of concrete, steel yield strength, diameter-to-thickness ratio, steel ratio and confining coefficient are prescribed in different codes. These limitations are compared and summarised in Table 1.

Table 1. Comparison of the limitations in the 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~ $90\sqrt{235/f_{ay}}$	20~100
a_a	0.04~0.16	–	0.04~0.20
ξ	–	0.03~0.3	–
	AISC-LRFD(99)	**EC4**	**JAN-AIJ(97)**
f_{ck}	26~65	25~60	
f_{ay}	≤ 415	235~355	
D/t_a	$\leq \sqrt{8E/f_{ay}}$	$\leq 90\sqrt{235/f_{ay}}$	$\leq 35280/f_{ay}$
a_a	≥ 0.04	–	–
ξ	–	–	–

$\xi = 1.5 A_a f_{ay} / A_c f_{ck}$, f_{ck} is the 150 mm cube compressive strength of concrete; f_{ay} – the yield strength of steel tube, A_a, A_c – areas of steel tube and concrete respectively, a_a – steel ratio; E – the elasticity modulus of steel tube.

Many researchers carry out tests which they then compare with a code or their own particular theory. Few look at others' test results. This is partly because it is difficult and tedious to gather the information together. This paper, and its associated website (*http://web.ukonline.co.uk/asccs2*), collects together information for 1303 composite column tests and compares the test results with EC4; some typical graphs are also included in this paper. It is hoped that other researchers will compare their theories with these data. The data cover static tests.

3. Databases

The data collected in the database on the website (*http://web.ukonline.co.uk/asccs2*) is subdivided into columns of "circular" and "rectangular' (mainly square) cross-section and into "short" (defined in the paper as $L/D(B) \leq 4$, Fig. 1) and "long" ($L/D(B) > 4$, Fig. 2) columns "with" and "without" moment. The source of the data is taken from Baochung and Hiroshi (2003), Chung *et. al* (2001), DL/T5085 (1999), Eurocode 4 (2005), Goode (1989), Goode (2007), Gopal and Manoharan (2003), Guolin and Zhong (2006), Han (2000), Han and Tao (2003), Han and Yang (2003), Han and Yao (2002), Han and Yao (2003), Han *et al.* (2004), Mursi *et al.* (2003), Kuranovas (2006), Kuranovas and Kvedaras (2007), Zhong (1996), Zhong (1999), Zhang and Zhong (1998), Zhang and Zhong (1999).

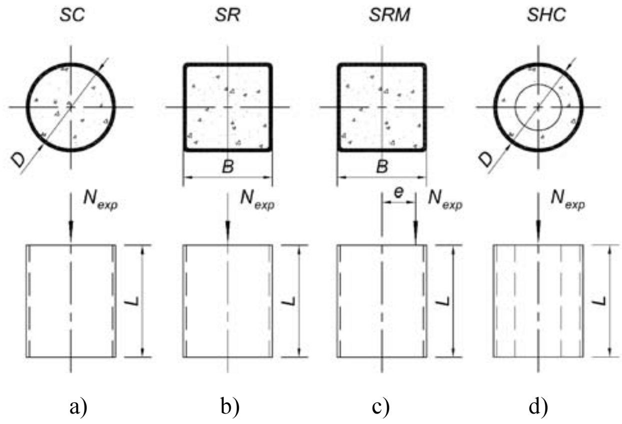

Fig. 1. Short CFST stub columns $L/D(B) \leq 4$: a) circular CFST with no moment, b) square CFST with no moment, c) square CFST with moment, d) hollow CFST with no moment

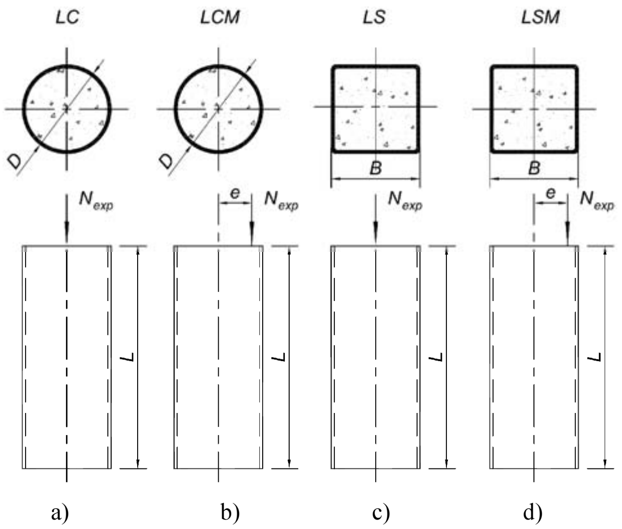

Fig. 2. Long CFST slenderness columns $L/D(B) > 4$: a) circular CFST with no moment, b) circular with moment, c) square CFST with no moment, d) square CFST with moment

The information required and reported for each test is: outer diameter (*D*) if circular cross-section, or breath (*B*) and depth (*H*) if rectangular; the thickness (t_a) of the steel tube; the steel properties (f_{ay}) and, for slenderness columns, modulus of elasticity (E_a); the concrete properties (concrete yield strength (f_{cyl}), (f_{ck} in EC4)) and, for long columns, its secant modulus of elasticity (E_c) to $0.4 f_{ck}$)); 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. The maximum lateral deflection at mid-height is also given when this has been reported by the researchers.

If E_a was not given, it was assumed to be 200 GPa. If concrete cube strength (f_{cu}) would be given, the cylinder strength was taken $0.8 f_{cu}$. If E_c equation was not provided it was calculated from the (Xinbo *et al*. 2006) $E_c = 0.0095 (f_{cyl} + 8)^{1/3}$ GPa, where f_{cyl} is in MPa.

4. Analysis of test result and comparing with EC4

EC4 requires the characteristic concrete cylinder strength, f_{ck}, to be at least 20 MPa and not more than 50 MPa "unless its use is appropriately justified". For thin walled section EC4 also includes a "local buckling criteria". However, all tests have been compared with ec4 regardless of these limitations. The EC4 design equations are given and discussed by Douglas *et al.*

The member has sufficient resistance if for both axes:

$$\frac{N_{Ed}}{N_{pl,Rd}} \le 1, \tag{1}$$

where $N_{pl,Rd}$ – plastic resistance to compression,

$$N_{pl,Rd} = A_a \eta_2 f_{ya} / \gamma_{Ma} + A_c f_{ck} \left(1 + \eta_1 (t_a / D_a)(f_{ay} / f_{ck})\right) / \gamma_c, \tag{2}$$

where A_a and A_c are the cross-sectional area of the structural steel and concrete; f_{ya} and f_{ck} are their characteristic strengths in accordance with EC3 and EC2; γ_{Ma} and γ_c are partial safety factors at the ultimate limit states; t_a – the wall thickness of the steel tube; η_1 and η_2 – coefficients; the other symbols are defined above.
The eccentricity of loading e is defined as:

$$M_{\max Ed} / N_{Ed}. \tag{3}$$

The values of η_1 and η_2 are:

$$\eta_1 = \eta_{10} (1 - 10e / D_a), \quad \eta_2 = \eta_{20} + (1 + \eta_{20})(10e / D_a). \tag{4}$$

For $e > D_a / 10$, $\eta_1 = 0$ and $\eta_2 = 1.0$.
The values of η_{10} and η_{20}, when $e = 0$ may be taken as follows:

$$\eta_{10} = 4.9 - 18{,}5\bar{\lambda} + 17\bar{\lambda} \text{ (but } \ge 0\text{)},$$

$$\eta_{20} = 0.25 (3 + 2\bar{\lambda}) \text{ (but } \le 1.0\text{)}. \tag{5}$$

The non-dimensional slenderness for the plane of bending considered is given by:

$$\bar{\lambda} = \sqrt{N_{pl,Rd} / N_{cr}}. \tag{6}$$

The elastic critical load for the column length, N_{cr}, shall be calculated from:

$$N_{cr} = \pi^2 (EI)_e / L^2, \tag{7}$$

where $(EI)_e$ is effective elastic flexural stiffness of cross-sections, and L is buckling length of a column.

$$(EI)_e = E_a I_a + 0.8 E_{cd} I_c, \tag{8}$$

where I_a and I_c are moments of inertia of area for considered bending plane of the structural steel and the concrete; E_a – elasticity modulus for the structural steel.

$$E_{cd} = E_{cm} / \gamma_c, \tag{9}$$

E_{cm} – mean value of concrete elasticity modulus

When comparing EC4 with tests, the materials safety factors (γ_{Ma} and γ_{ca}) have been taken as unity and concrete modulus as 1.35 because $0.8E_{cd} = 0.8E_c / 1.35 = 0.6E_c$, to give the effective elastic flexural stiffness $(EI)_e$ of the cross-section, required for long columns, as $(EI)_e = E_a I_a + 0.6 E_c I_c$. For columns with an end moment, the ultimate strength comparison with EC4 is at the same axial load/moment ratio as in the test.

In Figs 3–4 dispersion of EC4 vs test and ratio test/EC4 vs concrete strength for SC elements are provided and Figs 5–6 provides dispersions of EC4 vs test and ratio test/EC4 vs concrete strength for SR elements, Figs 7–8 – for SRM, Figs 9–10 – for SCH elements.

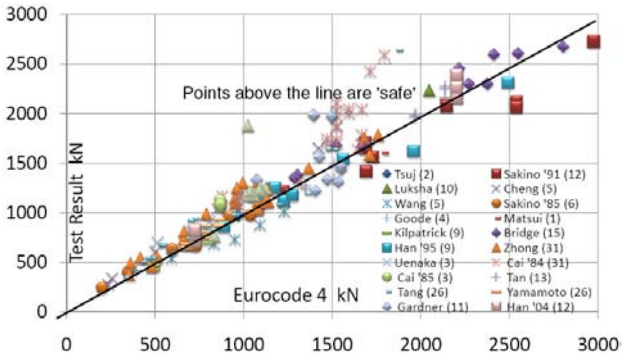

Fig. 3. Tests vs EC4 for short circular CFST columns without moment

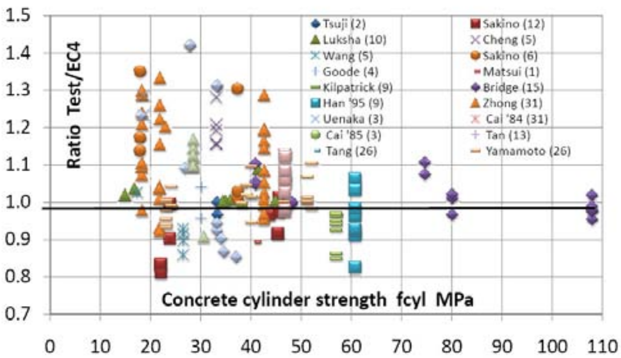

Fig. 4. Ratio test/EC4 vs concrete strength for short circular CFST columns without moment

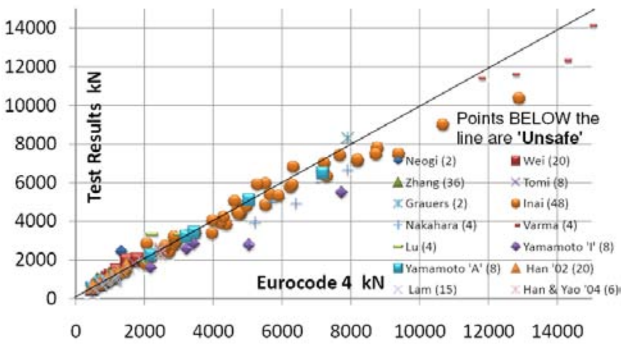

Fig. 5. Tests compared with EC4 for short rectangular CFST columns without moment

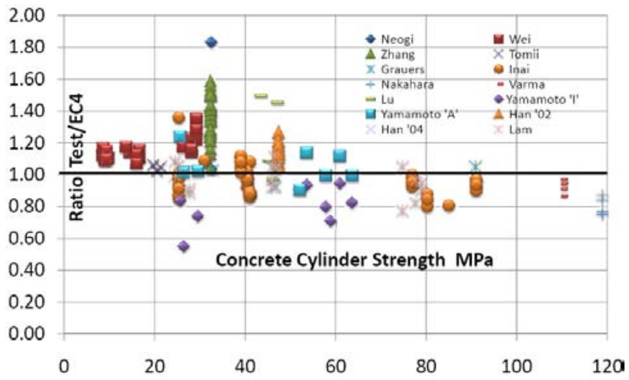

Fig. 6. Ratio test/EC4 vs concrete strength for short rectangular CFST columns without moment

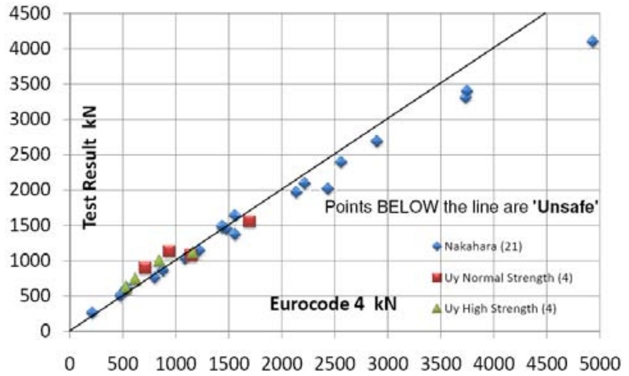

Fig. 7. Tests vs EC4 for short rectangular CFST columns with moment

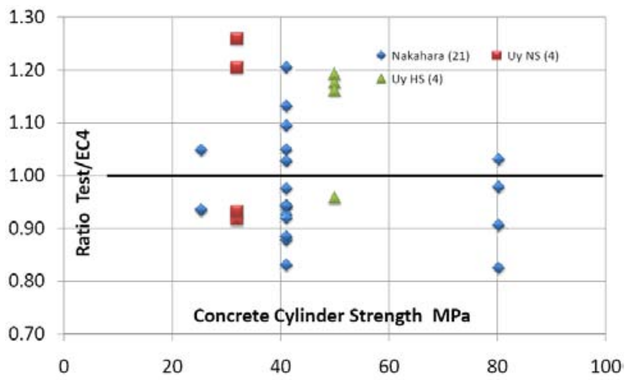

Fig. 8. Ratio test/EC4 vs concrete strength for short rectangular CFST columns with moment

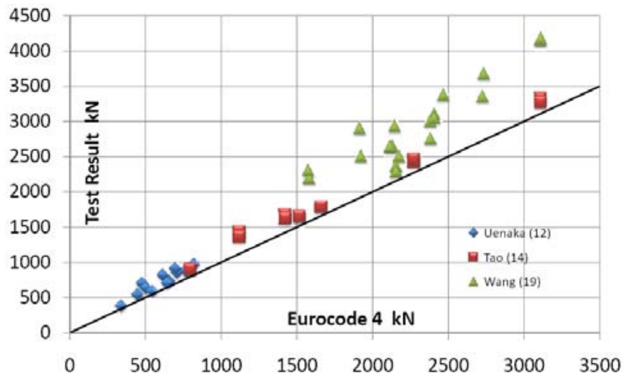

Fig. 9. Tests compared with EC4 for hollow short circular CFST columns without moment

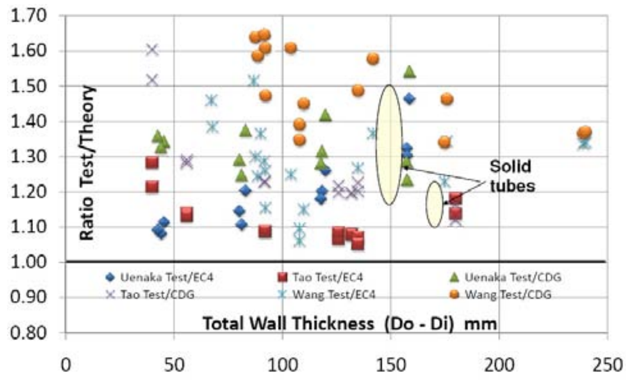

Fig. 10. Ratio test/EC4 and test/CDG (Eqn. 10) vs wall thickness for short hollow circular CFST

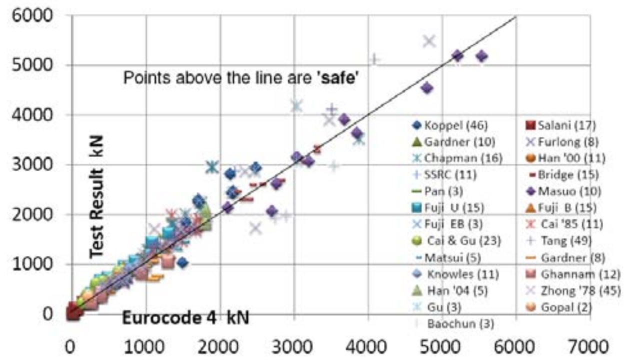

Fig. 11. Tests compared with EC4 for long circular CFST columns

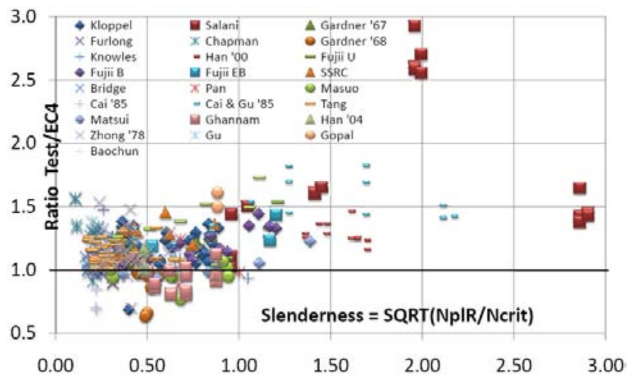

Fig. 12. Ratio test/EC4 vs slenderness for long CFST columns without moment

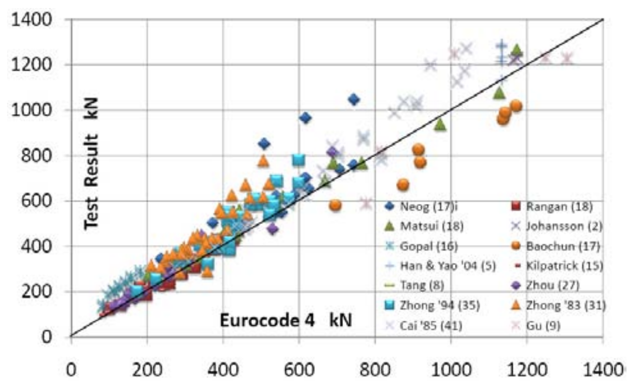

Fig. 13. Tests compared with EC4 for long circular CFST with moment

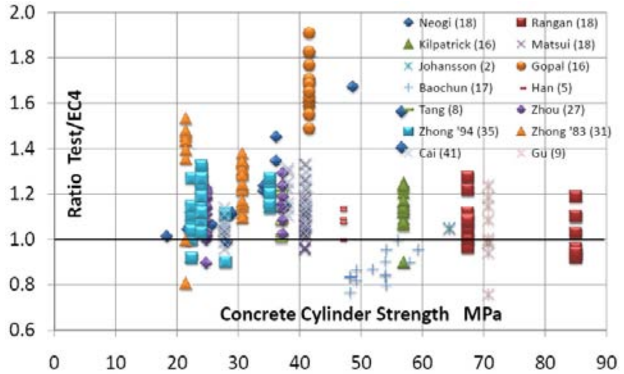

Fig. 14. Ratio Test/EC4 vs concrete cylinder strength for long circular CFST columns with moment

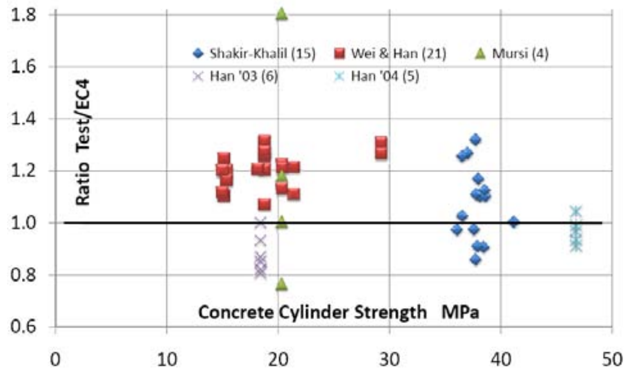

Fig. 18. Ratio test/EC4 vs concrete cylinder strength for long rectangular CFST columns with moment

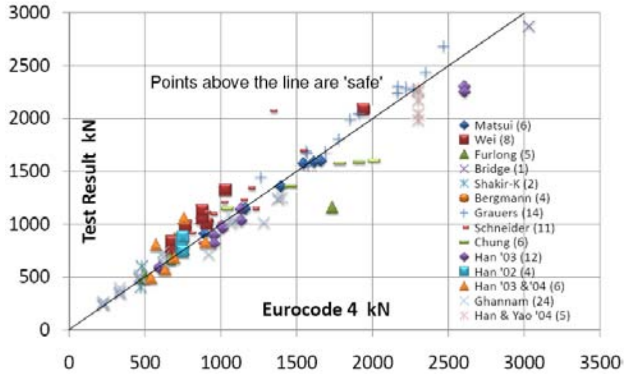

Fig. 15. Tests vs EC4 for long rectangular CFST columns without moment

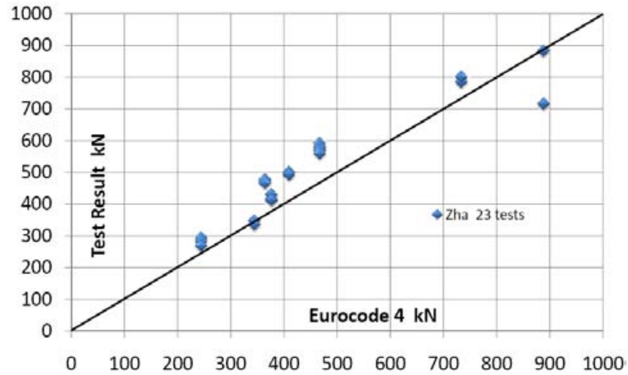

Fig. 19. Tests compared with EC4 for circular CFST columns with moment and preload

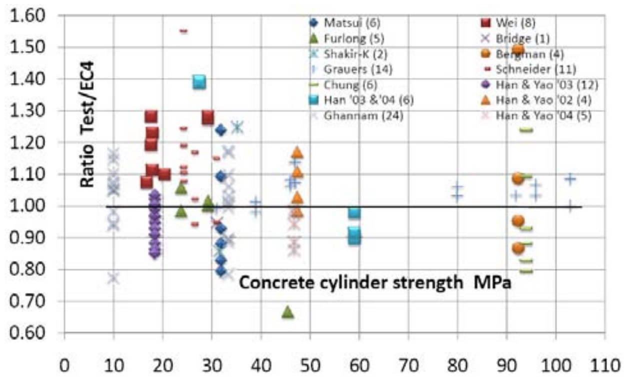

Fig. 16. Ratio test/EC4 vs concrete strength for long rectangular CFST columns without moment

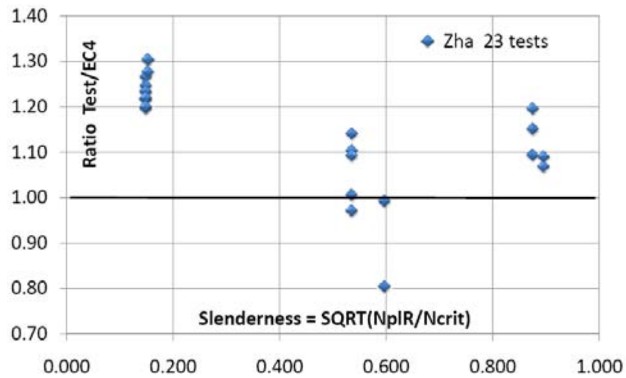

Fig. 20. Ratio test/EC4 against preload for long circular CFST columns with moment and preload

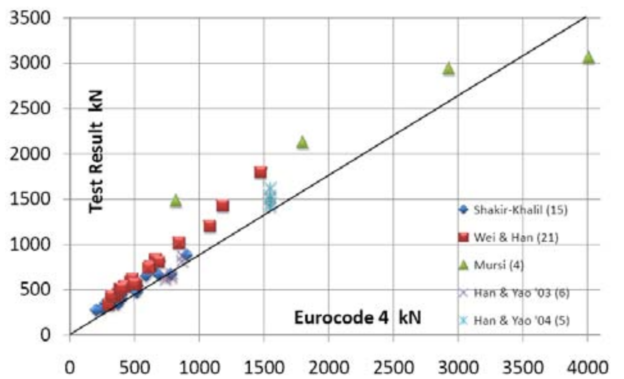

Fig. 17. Tests compared with EC4 long rectangular CFST columns with moment

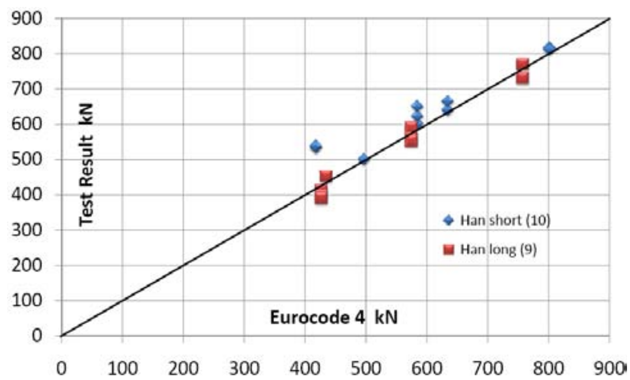

Fig. 21. Han's tests compared with EC4 for rectangular CFST columns with moment and preload

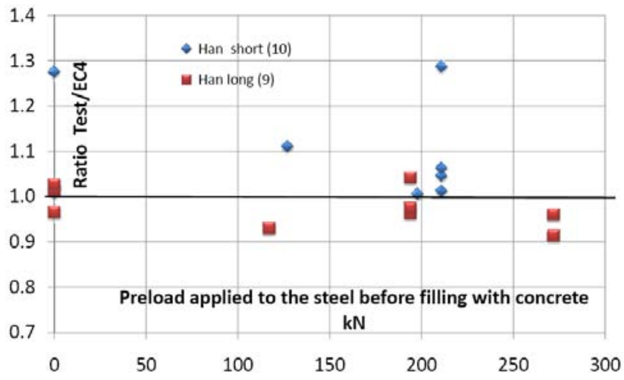

Fig. 22. Ratio test/EC4 vs preload for rectangular CFST columns with moment and preload

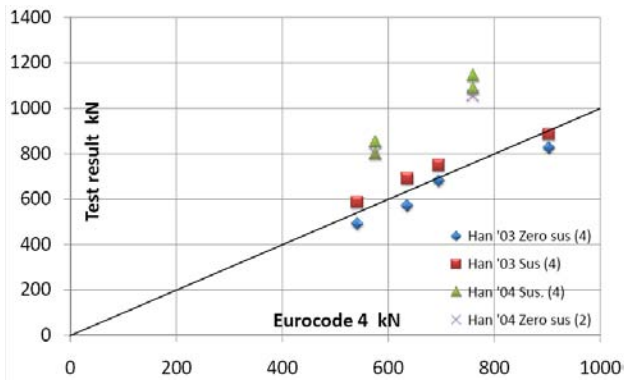

Fig. 23. Tests vs EC4 for rectangular CFST columns with sustained load

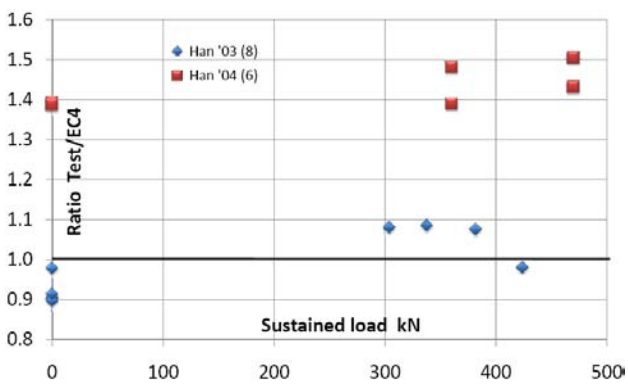

Fig. 24. Ratio test/EC4 vs sustained load for a long rectangular CFST columns

Representations of dispersions EC4 vs test and ratio test/EC4 vs concrete strength for slenderness elements are provided in Figs 11–18. Figs 11–12 represent values for LC and 13–14 for LCM elements. Dispersions of EC4 vs test and ratio test/EC4 vs concrete strength for LR and LRM are presented in Figs 15–16 and 17–18 respectively.

Figs 19–22 represent ratio test/EC4 vs preload for long circular and rectangular columns with moment and preload respectively. The dispersions of these ratios for long rectangular columns with sustained load are provided in Figs 23–24.

For circular cross-section columns there is good agreement between the test failure load and the Eurocode 4 calculation for both short and long columns with and without moment.

Short circular columns without moment the overall average test/EC4 from 243 tests is 1.07 with a standard deviation of 0.141.

Long circular columns without moment the overall average test/EC4 from 357 tests is 1.18 with a standard deviation of 0,250. The 17 tests by Salani and Sims, which were mortar filled, gave particularly high results (average = 1.80; SD = 0.609); excluding these tests the average of the other 340 tests is 1.17 with SD of 0.176.

Long circular columns with moment the overall average test/EC4 from 254 tests is 1.15 with a standard deviation of 0.111. However, Gopal's 14 tests with fibre RC filling are higher than this (average 1.68) and Baochun's 14 tests all gave unsafe values (Av. test/EC4 = 0.87). Excluding both Gopal and Baochun's tests gives: average (226 tests) = 1.23 with a standard deviation = 0.113.

Short hollow circular section columns the 26 tests have average test/EC4 of 1.16 with SD of 0.100.

For rectangular cross-section columns of agreement is good except when the concrete cylinder strength was greater than 75 MPa (strength greater than 50 MPa is not allowed in EC4), when many tests failed below the strength predicted by EC4.

Short rectangular section columns without moment the average test/EC4 from all the 185 tests is 1.09 with standard deviation 0.201. However, for higher strength concrete ($f_{cyl} > 75$ MPa), and thus columns of greater strength, the test results are lower than the EC4 approach predicts; for the 30 tests, where $f_{cyl} > 75$ MPa, the average test/EC4 is 0.91 with standard deviation 0,080.

Short rectangular (square) columns with moment the average test/EC4 of 29 tests is 1.01 with a standard deviation of 0.122.

Long rectangular columns without moment the overall average test/EC4 from 108 tests is 1.04 with a standard deviation of 0.143. The 17 tests with a concrete strength greater than 75 MPa did not show any reduction in the strength predicted by EC4; average test/EC4 being 1.03, SD = 0.164.

Long rectangular with moment the average test/EC4 from 51 tests is 1.10 with SD of 0.181.

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/EC4 for the 23 circular columns (11 short, 12 long) being 1.15 (SD 0.123) and for the 19 rectangular (10 short, 9 long) being 1.03 (SD 0.099).

Sustained load 8 tests by Han et al (2004) had an average sustained load of between 53% and 63% of their capacity for 120 or 180 days before being loaded to failure, the average test/EC4 is 1,25, which was higher than their 6 comparison tests without sustained load (average 1.08).

5. Structural behaviour

5.1. Constitutive relationship of steel

A typical stress σ_i – strain ε_i relationship for steel used in civil engineering under 3D stress state is shown in Fig. 25 (Zhang and Zhong 1999). The following assumptions are made: a) the strain hardening is simplified by a straight line *cd*, b) the failure is considered to be a horizontal straight line *de*.

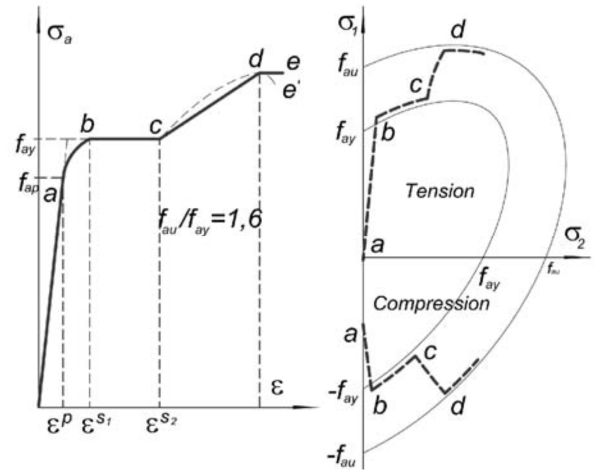

Fig. 25. Tension and compression steel $\sigma-\varepsilon$ relationships

The relation curve of stress intensity with strain intensity for steel under complex stress states is similarly to the stress strain relation curve under simply tension. There are 5 stages: elastic, elasto-plactic, plastic, strengthening and damage (Zhong 1996).

The equations of stress and strain are as follows:

$$\sigma_i = \frac{\sqrt{2}}{2}[(\sigma_{11}-\sigma_{22})^2 + (\sigma_{22}-\sigma_{33})^2 + (\sigma_{33}-\sigma_{11})^2 + (\sigma_{12}^2+\sigma_{23}^2+\sigma_{31}^2)]^{1/2}.$$

$$\varepsilon_i = \frac{\sqrt{2}}{3}[(\varepsilon_{11}-\varepsilon_{22})^2 + (\varepsilon_{22}-\varepsilon_{33})^2 + (\varepsilon_{33}-\varepsilon_{11})^2 + \frac{3}{2}(\varepsilon_{12}^2+\varepsilon_{23}^2+\varepsilon_{31}^2)]^{1/2}. \qquad (10)$$

In the elastic range, the proportional limit of steel $f_{ap} = 0.8 f_{ay}$; the Poisson's ratio $\nu_a = 0.283$; the elasticity modulus $E_a = 2.06 \times 10^5$ N/mm^2. In the elasto-plastic range, the tangent modulus of steel $E_a^t = \frac{(f_{ay}-\sigma_i)\sigma_i}{(f_{ay}-f_{ap})f_p} E_a$; the Poisson's ratio $\nu_a^t = 0.167\frac{(\sigma_i - f_{ap})}{(f_{ay}-f_{ap})} + 0.283$. f_{ap}, f_{ay} and f_{au} are proportional limit, yield stress and ultimate tensile strength respectively.

The constitutive relationship for the steel can be expressed as follows:

$$\{d\sigma_{ij}\} = [D]_s \{d\varepsilon_{ij}\}, \qquad (11)$$

where $[D]_s$ is the stiffness matrix for the steel, $[D]_s$ – the elastic stiffness matrix $([D]_e)$ in the elastic range, and in the plastic range is $[D]_{ep} = [D]_e - [D]_p$, where $[D]_p$ – the plastic stiffness matrix.

In the plastic range $\varepsilon_i^{s2} = 10\varepsilon_i^{s1}$, $\varepsilon_i^{s3} = 100\varepsilon_i^{s1}$; while in the strain hardening range, $f_{au}/f_{ay} = 1.6$.

5.2. Constitutive relationship of concrete core

There are many theories to describe the behaviour of concrete under triaxial compression. The constitutive relationship for concrete core of CFSTs is expressed using plastic-fracture theory in which the strains consists of elastic, plastic and fracture strains. Under 3D compression, total strain is:

$$d\varepsilon_{ij} = d\varepsilon_{ij}^{el} + d\varepsilon_{ij}^{pl} + d\varepsilon_{ij}^{fr}, \qquad (12)$$

where superscripts *e*, *p*, *f* mean for elastic, plastic and fracture strains respectively; *l* and *r* mean for longitudinal and radial strains respectively.

Typical constitutive relationships of concrete core 3D, 2D and uniaxial stress state are presented in Fig. 26 (Zhang and Zhong 1998).

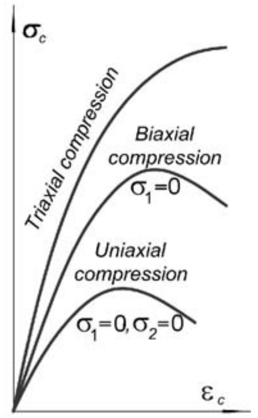

Fig. 26. Concrete $\sigma_c - \varepsilon_c$ relationships for uniaxial, 2D and 3D stress state

(2)

The constitutive relationship for concrete core in a 3D stress state can be expressed using plastic-fracture theory as follows:

$$\{d\sigma_c\} = [D]_c \{d\varepsilon_c\}, \qquad (13)$$

where $[D_c]$ – a 6×6 stiffness matrix.

There are 6 unknown parameters in this equation and they can be obtained by regression of experimental load-strain curves for concentrically loaded CFSTs.

5.3. Structural behaviour of CFSTs

The structural behaviour of CFST elements are considerably affected by the difference between the Poisson's ratios of the steel tube and concrete core. In the initial stage of loading, the Poisson's ratio for the concrete is lower than that of steel. Thus, the steel tube has no confining effect on the concrete core. As longitudinal strain increases, the lateral expansion of concrete gradually becomes greater than expansion of steel tube (Fig. 27). At this stage, the concrete core becomes triaxially and steel tube biaxially stressed (Kuranovas, Kvedaras 2007) (Fig. 28). The steel tube under a biaxial state cannot sustain the normal yield stress, causing a transfer of load from tube to the core. The load transfer mechanism is similar, square and circular CFST elements. In the first stage of loading the steel tube sustains most of the load until it yields (point a in Fig. 29). At this point (a) there is a load transfer from steel tube to the concrete core. The steel tube exhibits a gradual decrease in load sharing until the concrete reaches its maximum compressive strength (a to b). After this stage of loading (point b), there is redistribution of load from concrete core to the steel tube. At this point (b) the steel exhibits a hardening behaviour with almost the same slope as in uniaxial stress-strain hardening relationship (E_t).

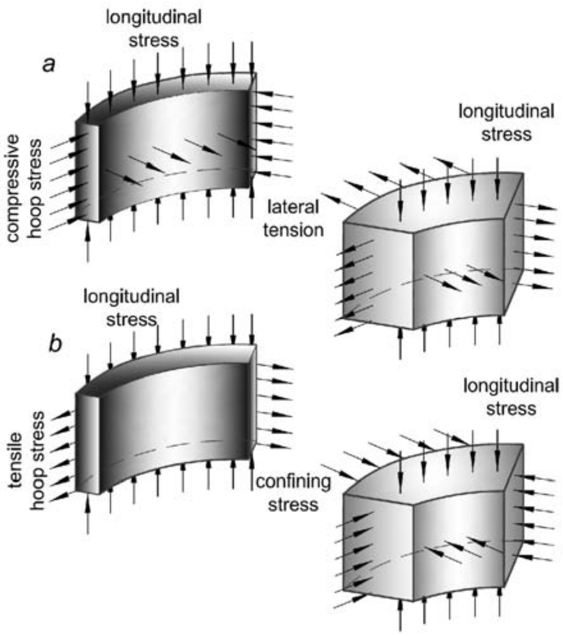

Fig. 27. Stress condition in steel tube and concrete core at different stages of loading: $\nu_a > \nu_c$ (a), $\nu_a < \nu_c$ (b)

The typical $\sigma - \varepsilon$ relationship shown in Fig. 6 consists of elastic (*oa*), elastoplastic (*ab*), and hardening (*bcd*) stages. Having such diagrams, it is easy to find the elasticity modulus E_{ac} and hardening modulus $E_{ac}^{'}$ of CFST element.

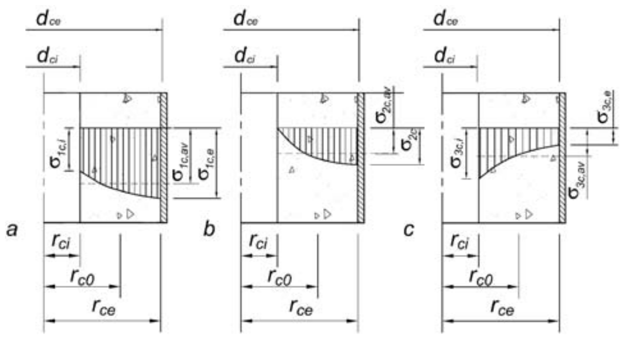

Fig. 28. Distribution of stresses in H-CFST element

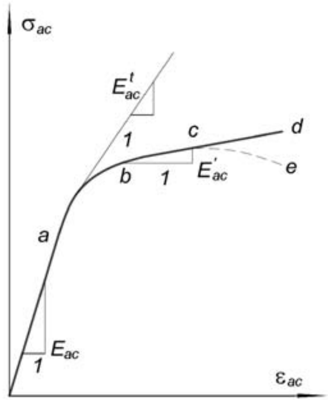

Fig. 29. $\sigma_{ac} - \varepsilon_{ac}$ relationship

Even though the load transfer mechanism in circular and square CFST is similar, the maximum confined compressive stress of concrete core in circular columns is higher than square column. This can be explained in terms of a larger confining effect of circular steel tubes, which is described in following sections.

6. Load-bearing capacity of H-CFSTs by other methods

EC2, EC4 and other sources provide design procedures and recommendation only for enhanced, CFST with solid concrete core elements. For hollow CFST elements no design recommendations are provided because of lack information, analyzis and test results.

One of few methods for determining load-bearing capacity is the "Unified theory" (Zhong 1996, 1999; Zhang and Zhong 1999) and, according to it, the member is considered as a unified body.

6.1. Unified theory

The content of "Unified theory" are: the concrete-filled steel tube is regarded as an unified body, which is a composite material, and its behaviour is changed with the change of physics parameters of materials, geometrical parameters of members, types of cross-sections and stresses states. The changes are continually, relatively, while the design is unified. In a word, the behaviour of CFSTs have unification, continuity and relativity.

From Unified theory, a unified design formula of CFSTs is produced. It can be used to design all of the members with different cross-sections. It makes a convenience of design work. And it is beneficially to draw up a unified standard for various CFST members.

Unified design formulas are provided as follows (Zhang and Zhong 1999):

when

$$\frac{N}{\varphi A_{ac}} \geq 0.2\sqrt{1-\left(\frac{T}{T_0}\right)^2-\left(\frac{V}{V_0}\right)^2} f_{ac}\,, \qquad (14)$$

$$\left[\sqrt[5]{\frac{N}{1.4\varphi N_0}+\frac{\beta M}{1.071M_0\left(1-0.4N/N_{ex}\right)}}\right]^7+\left(\frac{T}{T_0}\right)^2+\left(\frac{V}{V_0}\right)^2\le 1, \tag{15}$$

when

$$\frac{N}{\varphi A_{ac}}<0.2\sqrt{1-\left(\frac{T}{T_0}\right)^2-\left(\frac{V}{V_0}\right)^2}\,f_{ac}, \tag{16}$$

$$\left[\sqrt[5]{\frac{N}{1.4\varphi N_0}+\frac{\beta M}{M_0(1-0.4N/N_{\exp})}}\right]^7+\left(\frac{T}{T_0}\right)^2+\left(\frac{V}{V_0}\right)^2\le 1, \tag{17}$$

when axial force is tension

$$\left[\sqrt[5]{\frac{N}{N_t}+\frac{M}{M_0}}\right]^7+\left(\frac{T}{T_0}\right)^2+\left(\frac{V}{V_0}\right)^2\le 1, \tag{18}$$

where N, M, T and V are applied axial force, moment torsion and shear force, respectively. Relations between N/N_0; M/M_0 and T/T_0 ratios for CFSTs are presented in Fig. 30 (Zhang and Zhong 1999).

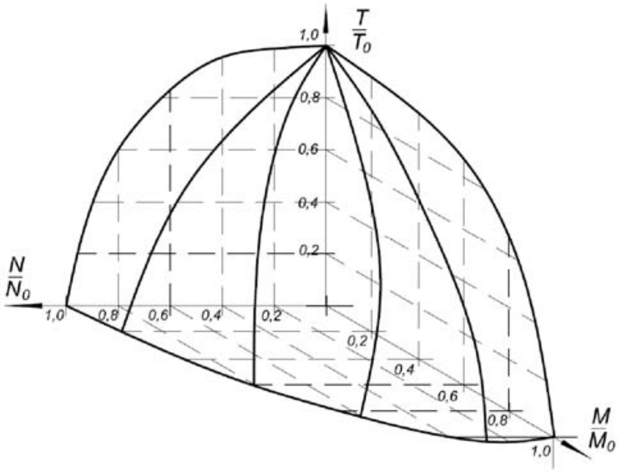

Fig. 30. Volume surface of *N-M-T* relations

There are 4 terms equations. When $T=0$, these formulas are changed to 3 terms equations; when $T=V=0$, it will be changed to 2 terms. And when it is axial compression or axial tension design formulas.

No matter what forms of members, solid or hollow sections or various cross-sections, these formulas can be used to design; the denominators of these formulas are resisting forces of members, which should be taken parameters for various members only. Hence, it is very conveniently for design.

Resisting axial compression

$$N_0 = A_{ac}f_{sc}, \tag{19}$$

resisting axial tension

$$N_t = k_1 A_a f, \tag{20}$$

resisting bending moment

$$M_0 = \gamma_M W_{ac} f_{ac}, \tag{21}$$

resisting torsion moment

$$T_0 = \gamma_T W_{ac}^T f_{ac}^v, \tag{22}$$

resisting shearing force

$$V_0 = \gamma_V A_{ac} f_{ac}^V, \tag{23}$$

where geometrical parameters of cross-section A_{ac}, A_a, W_{ac} and W_{ac}^T are total area of member of cross-section, area of steel tube, bending and torsion section modulus, respectively. It is a difference for various cross-sections.

The physical parameters f_{ac}, f_{ac}^V and f are composite design strength of compression, shearing of CFST and design compressive strength of steel, respectively. Coefficient k_1 for solid member is equal to 1.1, for hollow member 1.0.

For latticed members consisted of 2, 3 and 6 CFSTs, bearing capacity in plain should be calculated by following formula (Zhong 1996, 1999; Zhang and Zhong 1999):

$$\sqrt[7]{\left(\frac{N}{\varphi N_0}+\frac{\beta M}{M_0(1-\frac{\varphi N}{N_{ex}})}\right)^5+\left(\frac{V}{V_0}\right)^2}\le 1. \tag{24}$$

The composite compression design strength:
for solid cross-section

$$f_{ac}^y=\left(1.212+Bk_2\xi_0+Ck_2^2\xi_0^2\right)f_{ck}^0,\quad f_{ac}=\left(1.212+Bk_2\xi_0+Ck_2^2\xi_0^2\right)f_c, \tag{25}$$

for circular hollow cross-section

$$f_{ac}^k = f_{ac}-k_H\xi'\psi f_c,\quad B=0.1759f_y/235+0.974,\quad C=-0.1038f_{ck}^0/20+0.309, \tag{26}$$

for octagonal cross-section:

$$B=0.1401f_y/235+0.778,\quad C=-0.07f_{ck}^0/20+0.0262, \tag{27}$$

for square and rectangular cross-section:

$$B=0.131f_y/235+0.723,\quad C=-0.07f_{ck}^0/20+0.0262, \tag{28}$$

where ξ, ξ_0 is design confining index of solid member, $\xi_0 = af/1.1f_{ck}$; $\xi_0 = af/f_c$; a – steel ratio of solid member, $a = A_a/A_c$; ξ' – design confining index of hollow member, $\xi' = a'f/f_c$; a' – steel ratio of hollow member; ψ – hollowness ratio, $\psi = A_H/\left(A_{ac}-A_a\right)$; A_{ac} – total area of member; A_a – area of steel tube; A_H – area of hollow part; f – compression strength of steel; and f_c – compression strength of concrete.

For hollow CFST member, the compression strength of concrete should be taken $1.1f_c$ owing to the concrete maintained by steam pouring. The compression strength can be enhanced by 10%.

The composite shearing design strength:

$$f_{sc}^{V} = \left(0.385 + 0.25\sqrt{a^3}\right)\xi_0^{1/8} f_{ac} . \quad (29)$$

The buckling coefficient of axial compression is:

$$\varphi = k_0 \varphi_0 , \quad (30)$$

where coefficient φ_0 is buckling coefficient of circular solid CFST member, as shown in Table 2. The values of k_0 for solid and hollow sections are: for circular 1.0, for 16-side member 0.95, for octagon – 0.9 and for square and rectangular – 0.85. φ_0 is presented in Table 3. The coefficient of plastic development γ_M, γ_T and γ_V are listed in Table 4.

Table 2. Coefficients k_2 and k_H

Coef.	Cross-sections					
	Circular and 16-side	Octagonal	Square			
			a			
			0.05	0.10	0.15	0.20
k_2	1.0	0.8	0.74	0.73	0.72	0.71
k_H	0.2	0.4	0.5			

Table 3. Coefficient φ_0

λ	30	40	50	60	70	80
S235	0.989	0.972	0.946	0.912	0.860	0.819
S355	0.987	0.966	0.935	0.865	0.844	0.783
	90	100	110	120	130	140
S235	0.760	0.692	0.617	0.521	0.444	0.383
S355	0.712	0.693	0.541	0.455	0.387	0.334

Table 4. Coefficients γ_M, γ_T and γ_V

Coef.	For all types of solid cross-sections	For all types of hollow cross-sections
γ_M	$-0.4832\xi + 1.9264\sqrt{\xi}$	$-0.4832\xi + 1.9264\sqrt{\xi} \le 1$
γ_T	$-0.4701\xi + 1.8913\sqrt{\xi}$	$-0.4701\xi + 1.8913\sqrt{\xi} \le 1$
γ_V	$-0.2953\xi + 1.2981\sqrt{\xi}$	$-0.2953\xi + 1.2981\sqrt{\xi} \le 0.9$

In Fig. 31 it can be noticed that for solid CFST there is no descending stage in $N_{ac} - \varepsilon_{ac}$ diagram, but for H-CFST such stage exists, and the plastic stage is shortened while hollow ratio is increased, finally, the brittle damage occurs.

The descending stage occurs in H-CFSTs, because the strength indexes at the point of elasto-plastic stage and the elasticity modulis are lower than that of solid one. And they are decreasing with increasing the hollowness ratio ψ.

Results show: the higher the steel ratio α_a, load-bearing capacity and the bigger slope of plastic-hardening stages; the smaller hollowness ratio ψ, the higher load-bearing capacity and the more similar their behaviour to solid CFST members; in the only case, when the hollowness is big and steel ratio is small, there is descending stage on the $N_{ac} - \varepsilon_{ac}$ curve; the failure of hollow CFST members starts from the inside surface of the concrete tube because the concrete is there in biaxial compression; the above calculated curves are very close to the test curves for both solid and hollow CFST elements.

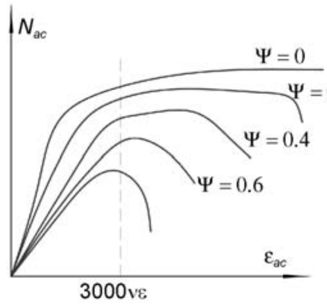

Fig. 31. $N_{ac} - \varepsilon_{ac}$ relationship related with hollowness ratios ψ

On the elastic range of H-CFSTs under axial compression, both the concrete core and the steel tube are under uniaxial stress state. There is no confinement from the steel to the concrete core. The elasticity modulus at this stage with different cross-section geometries is expressed as follows:

$$E_{ac} = f_{ac}^{p} / \varepsilon_{ac}^{p} , \quad (31)$$

where f_{ac}^{p} and ε_{ac}^{p} are the average proportional limit stress and strain of H-CFSTs, respectively.

$$f_{ac}^{p} = \left(0.192 f_{ay} / 235 + 0.488\right) f_{ac}^{y} , \quad (32)$$

$$\varepsilon_{ac}^{p} = 0.67 f_{ay} / E_a . \quad (33)$$

Different cross-section geometries have different f_{ay}^{y} and E_{ac}.

In the elasto-plastic range, the tangent modulus of an H-CFST element is characterised by the following equation:

$$E_{ac}^{t} = \frac{\left(A_1 f_{ac}^{y} - B_1 \bar{\sigma}\right)\bar{\sigma}}{\left(f_{ac}^{y} - f_{ac}^{p}\right) f_{ac}^{p}} E_{ac} , \quad (34)$$

where

$$A_1 = 1 - \frac{E_{ac}^{'}}{E_{ac}} \left(\frac{f_{ac}^{p}}{f_{ac}^{y}}\right)^2 , \; B_1 = 1 - \frac{E_{ac}^{'}}{E_{ac}} \left(\frac{f_{ac}^{p}}{f_{ac}^{y}}\right) . \quad (35)$$

In the hardening phase, the tangent modulus is:

$$E_{ac}^{'} = 400\xi - 150 . \quad (36)$$

6.2. Other methods

Goode (2007) proposes to calculate load-bearing capacity of H-CFST elements by Eq. 35:

$$N_{pl,R} = A_a f_y + A_c k_c f_{ck} , \quad (37)$$

where k_c – coefficient of increased concrete strength in centrifuged core, which can be calculated by Eq. 36:

$$k_c = 4.87 - 28.9d + 39.7t_s + 0.034 f_{ck} + 0.434 t_s / t_c + 0.1133 f_{ck} d + 55.6 d^2 , \quad (38)$$

where d, t_s, t_c – are external diameter of concrete core, thicknesses of steel tube and concrete core respectively.

A. K. Kvedaras (1999) proposes to calculate the strength of H-CFST as sum of forces acting composite cross-section (Eq. 39):

$$N_{pl,R} = N_{au} + N_{cu}, \tag{39}$$

where N_{au}, N_{cu} are load-bearing capacities of steel shell and concrete core correspondingly and can be determined by Eqs. 40, 41:

$$N_{au} = 1.074 f_y A_a, \tag{40}$$

$$N_{cu} = 1.32 f_{ck} A_c. \tag{41}$$

C. D. Goode (1989) suggests evaluating ultimate load value of composite member by modified EC4 formula Eq. (40), which according author predict well load-bearing capacity of CFST member:

$$N_{pl,Rd} = 0.68 f_{ck} A_c + 6 f_y A_c t / (D - 2t). \tag{42}$$

Kuranovas (2006) proposes to determine ultimate load of H-CFST element with evaluation of stress redistribution in concrete core. For non-slender $L/D \leq 4$ elements ultimate load can be calculated by

$$N_{pl,R} = k f_c A_c + f_{a,y} A_a, \tag{43}$$

where k – coefficient taking into account the increase of strength.

As result of testing results, processing for k coefficient determination mathematical model of progression was derived.

$$k = 1 + m A_a / A_c - n A_a f_c / A_c, \tag{44}$$

where m = 5 – for one-layered, m = 7 – for double-layered elements, n = 0.1 – for one-layered, n = 0.09 – for double-layered elements.

Evaluation of all suggested Eqs. (37–44) is presented by Kuranovas (2006) and the results show that Eqs. (43–44) predict results with ultimate load for H-CFST elements with average value 1.02 of predict and test ratio and with variation coefficient of 0,03 value. And most of predicted results are less than experimental ones. Results obtained from Eqs. (39–41) give corresponding values of 1.02 and 0.08 correspondingly and very well predict strength of composite members. Eqs. 43, 44 evaluate the phenomenon of strength increase from multi-layering of concrete core and more precisely predict ultimate loads than other scientists suggested.

Fig. 32 shows dispersion of experimental results vs predicted values according to Eq. 43.

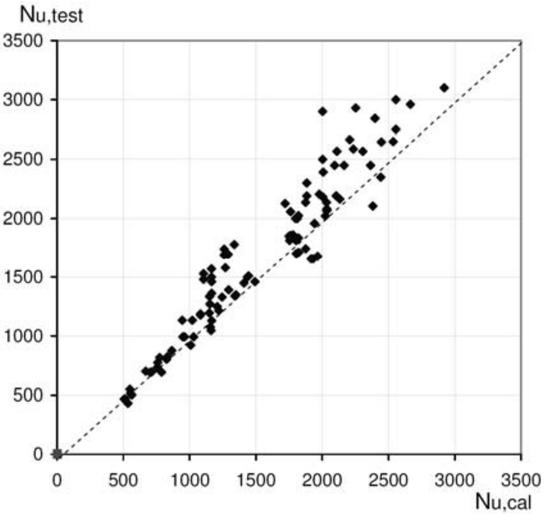

Fig. 32. Dispersion of experimental results vs predicted values

7. Conclusions

Investigations show that the behaviour of hollow CFST elements is more complicated than that of solid ones, because of complex stress states none of stresses in hollow concrete core are evenly distributed through the thickness of its cross-section.

At present it is a lack of information for H-CFSTs designing. Different approaches and design philosophies have been adopted in different design codes.

Eurocode 4 is a very good, and safe, predictor of strength for all types of circular cross-section CFST columns and could be safely used for concrete with cylinder strength up to 100 MPa.

For rectangular section CFST columns Eurocode 4 should be used with caution, when the concrete cylinder strength is greater than 75 MPa as the failure load in the majority of tests, when f_{cyl} > 75 MPa, was less than that predicted by the EC4 approach. (Note: EC4 limits the concrete strength to 50 MPa.) The factor 0,85 which is usually applied to the cylinder strength to relate it to the uniaxial strength in the 'stress block' is omitted, in EC4, for filled tubes, probably because of the confining effect of the tube. Omitting this factor for all sizes of tube and concrete strength seems very arbitrary and, for a greater safety, it is suggested that for rectangular section tubes this 0,85 factor should be included, when concrete with a cylinder strength greater than 75 MPa is used.

Pre-load of the steel tube, up to 60% of the capacity of the steel, before filling with concrete, seems to have had little effect on the strength of the column.

Sustained load of up to 63% of the column's capacity for up to 180 days did not reduce the strength of the 8 columns, when subsequently tested to failure.

The simplified 'k' factor method and second order analysis of Eurocode 4 gave similar results. For the 254 circular columns the average test/EC4 ratio by the 'k' factor method gave 1.15. And also 1.15 by the second order analysis; for the 51 rectangular columns the ratio was 1.11, by the 'k' factor method and 1.16 using the second order analysis.

The establishment of Unified theory provides a new research method and design method of CFSTs.

The Unified theory analyses the concrete-filled steel tube as a unified body, which is composite material and consists of steel tube and concrete core. Behaviour of this element changed with physical parameters of materials, geometrical parameters and the type of cross-section. The changes are continuous relatively, while the design is unified.

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

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BETONŠERDŽIŲ PLIENINIŲ VAMZDINIŲ KOLONŲ LAIKOMOJI GALIA

A. Kuranovas, C. D. Goode, A. K. Kvedaras, S. T. Zhong

Santrauka

Straipsnyje analizuojami 1303 betonšerdžių plieninių strypų bandinių eksperimentiniai duomenys. Duomenys lyginami su eurokode 4 pateiktais kompozitinių elementų laikomosios galios nustatymo metodais. Analizuojami šie betonšerdžių plieninių strypų bandinių tipai: pilnaviduriai ir tuščiaviduriai, apskrito ir stačiakampio skerspjūvio kolonos, kurių galuose veikia arba neveikia momentas, su iš anksto pridėta arba ilgalaike apkrova. Apskrito skerspjūvio kolonų laikomosios galios bandymų rezultatai atitinka skaičiavimų reikšmes, apskaičiuotas pagal eurokode 4 pateiktu metodu. Stačiakampio skerspjūvio elementų laikomosios galios reikšmių bandymo rezultatai puikiai atitinka teorines reikšmes, kai betono ritininis stipris nesiekia 75 MPa. Išankstinis elementų apkrovimas poveikio elementų laikomajai galiai beveik neturi. Taip pat nagrinėjami betonšerdžių elementų įtempių būvių pasiskirstymas, betono apspaudimo poveikis ir išilginių deformacijų ir įtempių kreivės. Pateikiama S. T. Zhong „Unifikuota teorija“, kuri nagrinėja kompozitinį elementą kaip visumą. Straipsnyje nagrinėjamos kompozitinio plieninio ir betoninio elemento stiprumo ir pastovumo sąlygos. Tokių elementų elgsena pagal teoriją priklauso nuo tuštumos santykio ir apspaudimo indekso, kurie grindžiami pilnavidurio elemento reikšmėmis. Jeigu tuštumos santykis lygus 0,4–0,5 ir daugiau, N-ε sąryšis yra kritimo stadijoje. Elgsenos stadijos keičiasi pagal tuštumos koeficientą.

Reikšminiai žodžiai: kompozitinės konstrukcijos, betonšerdžiai, plieniniai vamzdžiai, eurokodas 4, lyginimas, laikomosios galios analizė, betonšerdžiai plieniniai elementai, elgsena, įtempių būviai, Puasono koeficientas, tamprumo modulis.

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