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Behavior and calculation on concrete-filled steel CHS (Circular Hollow Section) beam-columns

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Abstract. A mechanics model is developed in this paper for concrete-filled steel CHS (circular hollow section) beam-columns. A unified theory is described where a confinement factor (ξ) is introduced to describe the composite action between the steel tube and the filled concrete. The predicted load versus deformation relationship is in good agreement with test results. The theoretical model was used to investigate the influence of important parameters that determine the ultimate strength of concrete-filled steel CHS beam-columns. The parametric and experimental studies provide information for the development of formulas for the calculation of the ultimate strength of the composite beam-columns. Comparisons are made with predicted beam-columns strengths using the existing codes, such as LRFD-AISC-1999, AIJ-1997, BS5400-1979 and EC4-1994.

Key words: composite columns; beam-columns; composite actions; constraining factor; concrete; design; hollow sections; columns; member capacity.

1. Introduction

Circular hollow section (CHS) steel tubes are often filled with concrete to form a composite column in modern building construction. Such kind of composite columns are well recognised in view of their high load carrying capacity, fire resistance, stiffness, ductility and energy absorption capacity, fast construction, as well as small cross section (ASCCS 1997, Shanmugam and Lakshmi 2001, Gourley *et al*. 2001).

In the past, there were a large number of research studies on concrete-filled steel CHS columns, such as Elchalakani *et al*. (2001), Furlong (1967), Gardner and Jacobson (1967), Han (2000a, 2000b), Johansson and Gylltoft (2001), Kato(1996), Kilpatrick and Rangan (1997), Kloppel and Goder (1957), Knowles and Park (1969), Luksha and Nesterovich (1991), Masuo *et al*. (1991), Matsui *et al*. (1995), Neogi *et al*. (1969), O'shea and Bridge (1997a, 1997b), Prion and Boehme (1994), Rangan and Joyce (1991), Sakino and Hayashi (1991), Sakino *et al*. (1985), Schneider

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(1998), Task Group 20 (1979), and etc. A total of 384 tests results, including 95 stub columns, 167 columns, 16 beams and 106 beam-columns were presented in these references. They will be used to calibrate the models proposed in this paper. The tests were with a wide range of parameters such as the confinement factor (ξ), the slenderness ratio (λ), the concrete strength, the steel yield stress and the load eccentricities.

Simple superposition of the strength of concrete and steel tubes was used traditionally to estimate the section capacity of concrete-filled steel tubular columns. Simple superposition tends to give reasonable estimation of ultimate section capacity but underestimate the ductility of the composite section. An efficient ductility of composite sections is very important especially under earthquake loading. The interaction between steel tube and concrete is the key issue to understand the behaviour of concrete-filled tubular members. Mechanics models were developed for concrete-filled SHS (Square Hollow Section) stub columns, columns and beam-columns by the authors (Han *et al.* 2001), whereas this paper addresses concrete-filled steel CHS (Circular Hollow Section) stub columns, columns and beam-columns.

A unified theory was presented in Han *et al*. (2001) where a confinement factor (ξ) was introduced to describe the composite action between the steel tube and filled concrete. The unified theory is adopted in this paper to develop mechanics models for concrete-filled steel CHS columns and beam-columns. The predicted column strength is compared with current 384 test results mentioned above, reasonable agreement was obtained. The load versus mid-span deflection relationship is established for concrete-filled steel CHS columns and beam columns theoretically. The predicted curves of load versus mid-span deflection are generally in good agreement with test results. A simplified model is developed for calculating the section capacity, the member capacity and the moment capacity of concrete-filled steel CHS members. Simplified interaction curves are derived for concrete-filled steel CHS beam-columns. Comparisons are made with predicted column strengths using LRFD (AISC 1999), AIJ (1997), BS5400 (1979) and EC4 (1994).

2. Mechanics model for concrete-filled CHS columns

2.1. Material properties

A typical stress-strain curve for steel can consist of five stages. Detailed derivations of the stress-strain relationship were given in Han *et al.* (2001). A summary is presented in Appendix I.

A typical stress-strain curve for confined concrete is shown in Fig. 1, where the confinement factor (ξ) is defined as (Han *et al*. 2001, Han and Huo 2003):

$$\xi = \frac{A_s \cdot f_{sy}}{A_c \cdot f_{ck}} = \alpha \cdot \frac{f_{sy}}{f_{ck}} \tag{1}$$

in which A_s is the cross-section area of steel tube; A_c is the cross-section area of concrete; $\alpha = A_s/A_c$, is defined as steel ratio; f_{sy} is the yield stress of steel tube, and f_{ck} is the compression strength of concrete. The value of f_{ck} is determined using 67% of the compression strength of cubic blocks. Detailed expressions are given in Han and Huo (2003). A summary is presented in Appendix II.

It can be seen from Fig. 1 that the higher the confinement factor (ξ), the higher the compression strength of confined concrete. It can also be seen from Fig. 1 that the higher is ξ, the more ductile is the confined concrete. The confinement factor (ξ), to some extent, represents the composite action between steel tubes and concrete core.

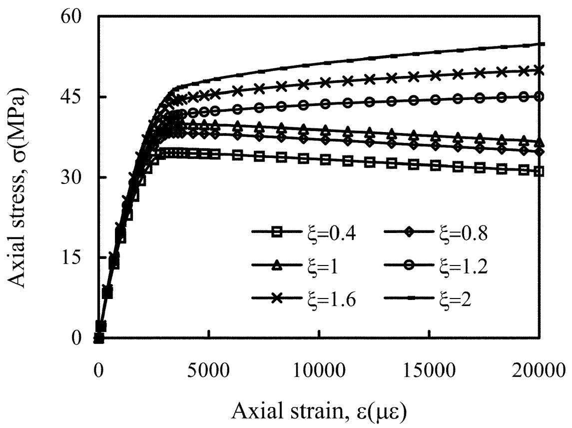


Fig. 1 σ versus ε relations of confined concrete (f_{ck} = 26.8 MPa)

2.2. Mechanics model for concrete-filled CHS beam-columns

2.2.1. Assumptions

A member subjected to compression is shown in Fig. 2 where N is the compression force, e is the load eccentricity and u_m is the mid-span deflection. When the load eccentricity (e) equals zero, the member under compression is called a column. Otherwise the member is called a beam-column, i.e., it is under combined bending and compression.

The load versus mid-span deflection relations can be established based on the following assumptions:

1) The stress-strain relationship for steel given in Appendix I is adopted for both tension and compression.
2) The stress-strain relationship for concrete given in Appendix II is adopted for compression only. The contribution of concrete in tension is neglected.
3) Original plane cross-sections remain plane.
4) The effect of shear force on deflection of members is omitted.
5) The deflection curve of the member is assumed as a sine wave.

2.2.2. Load versus deformation relations

According to the assumption No. 5, the deflection (u) of the member can be expressed as:

$$u = u_m \cdot \sin\left(\frac{\pi}{L} \cdot z\right) \tag{2}$$

where, z is the horizontal distance from the left support as defined in Fig. 2.

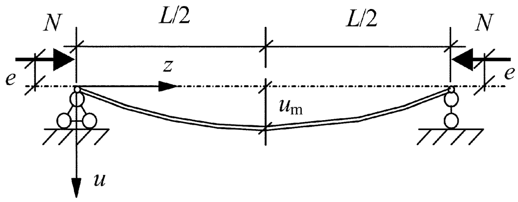


Fig. 2 A schematic view of a beam-column

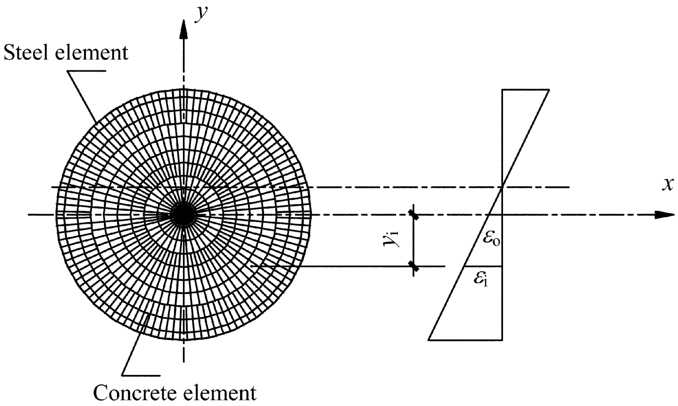


Fig. 3 Distribution of strains

The curvature (ϕ) at the mid-span can be calculated as:

$$\phi = \frac{\pi^2}{L^2} \cdot u_m \tag{3}$$

The strain distribution is shown in Fig. 3 where ε_o is the strain along the geometrical centre line of the section. The term ε_i is the strain at the location y_i as defined in Fig. 3. Along the line with $y = y_i$ the section can be divided into two elements (dA_{si} for steel and dA_{ci} for concrete) with unit depth. The strain at the centre of each element can be expressed as:

$$\varepsilon_i = \varepsilon_o + \phi \cdot y_i \tag{4}$$

The stress at the centre of each element (σ_{si} for steel or σ_{ci} for concrete) can be determined using the stress-strain relationship given in Appendices I and II. The internal moment (M_{in}) and axial force (N_{in}) can be calculated as:

$$M_{in} = \sum_{i=1}^{n} (\sigma_{si} \cdot x_i \cdot dA_{si} + \sigma_{ci} \cdot x_i \cdot dA_{ci}) \tag{5}$$

$$N_{in} = \sum_{i=1}^{n} (\sigma_{si} \cdot dA_{si} + \sigma_{ci} \cdot dA_{ci}) \tag{6}$$

According to the equilibrium condition,

$$M_{in} = M_{applied} \tag{7}$$

$$N_{in} = N_{applied} \tag{8}$$

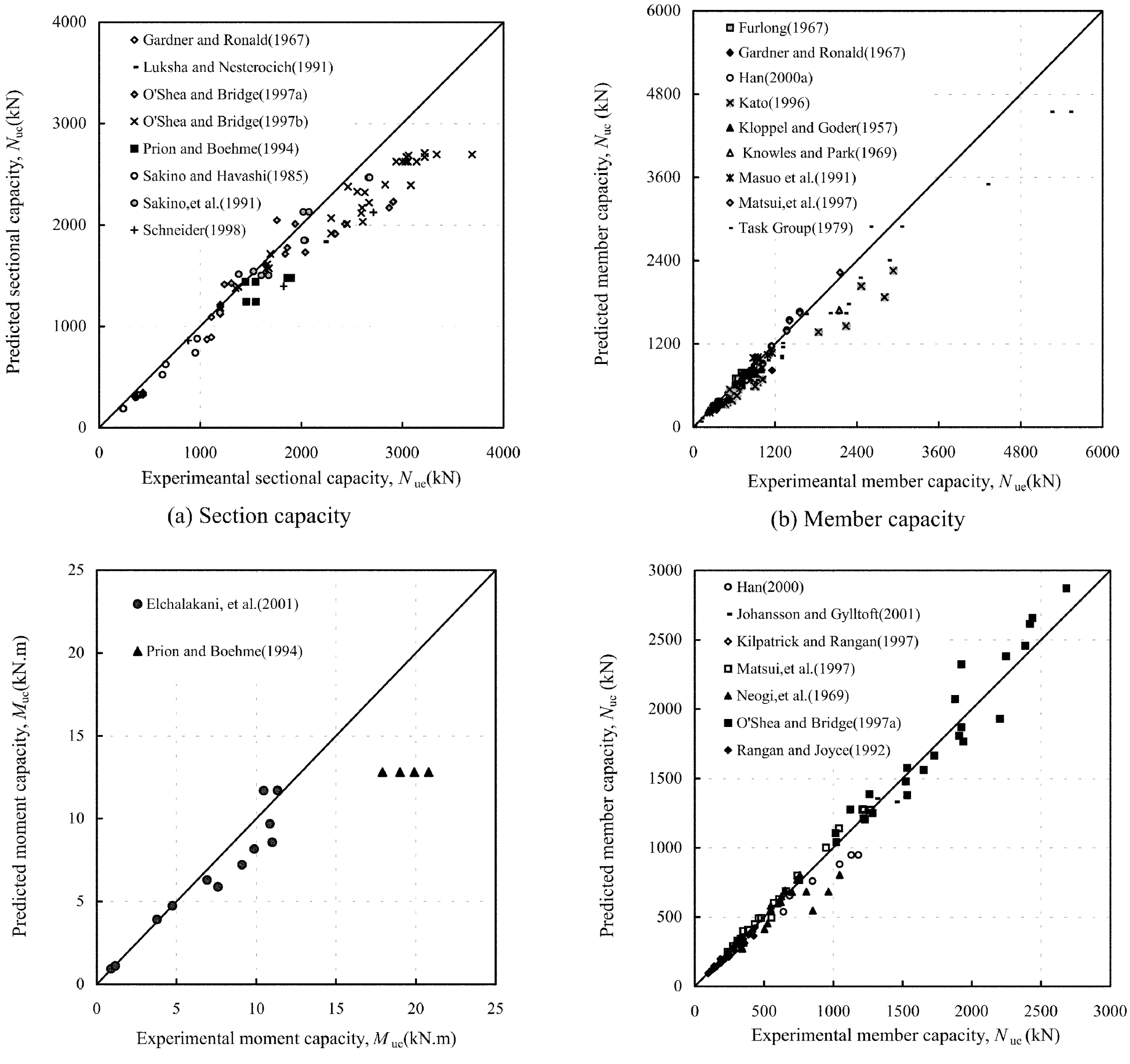
From the above equations, the load versus mid-span deflection relations can be established for a certain eccentricity (e).

In the calculations, a small arbitrary load eccentricity of L/1000, reflecting a nearly ideal straight in axis of the column, has been selected for the initial eccentricity (Han *et al.* 2001).

2.3. Comparisons between predicted and measured results

The column section capacity predicted using the mechanics model are compared with 95 stub column test results obtained from Gardner and Jacobson (1967), Luksha and Nesterovich (1991), O'shea and Bridge (1997a), O'shea and Bridge (1997b), Prion and Boehme (1994), Sakino and Hayashi (1991), Sakino *et al.* (1985) and Schneider (1998). Fig. 4(a) shows the comparisons, where a mean of 0.909 and a COV (coefficient of variation) of 0.095 are obtained.

Fig. 4(b) shows the comparisons between column member capacity predicted using the mechanics model and 167 column test results obtained from Furlong (1967), Gardner and Jacobson (1967), Han (2000a), Kato (1996), Kloppel and Goder (1957), Knowles and Park (1969), Masuo *et al.* (1991), Matsui



(a) Section capacity

(b) Member capacity

(c) Moment capacity

(d) Beam-column member capacity

Fig. 4 Comparison of column strength between theoretical model and test

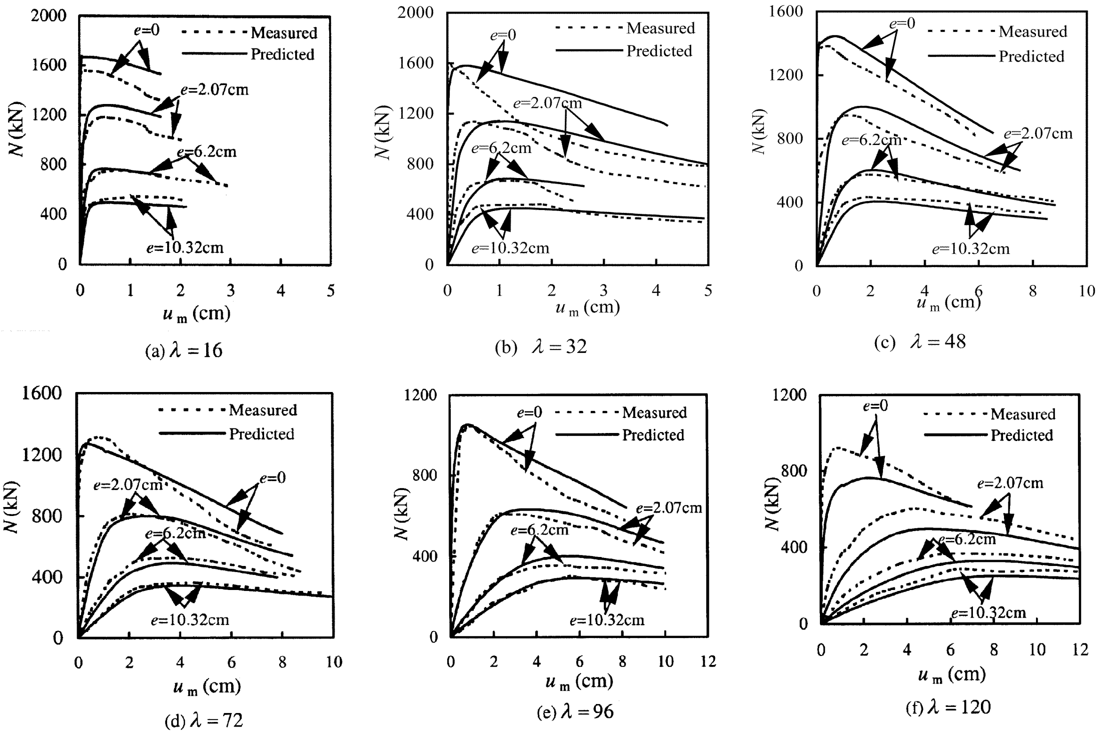


Fig. 5 Comparison of predicted load (N) versus mid-span lateral deflection (u_m) curves with test results by Matsui *et al.* (1995)

et al. (1995) and Task Group 20 (1979), where a mean of 0.905 and a COV of 0.115 are obtained.

The moment capacity predicted using the mechanics model are compared with 16 beam test results obtained from Elchalakani *et al.* (2001), Prion and Boehme (1994) in Fig. 4(c). Where a mean of 0.863 and a COV of 0.152 are obtained.

The member capacities of a total of 106 beam-columns obtained from Han (2000b), Johansson and Gylltoft (2001), Kilpatrick and Rangan (1997), Matsui *et al.* (1995), Neogi *et al.* (1969), O'shea and Bridge (1997a), Rangan and Joyce (1991), are compared with member capacity predicted using the mechanics model in Fig. 4(d). Where a mean of 0.976 and a COV of 0.094 are obtained.

The predicted load versus lateral deflection curves using the mechanics model are compared in Fig. 5 with the a set of experimental values obtained by Matsui *et al.* (1995), parameters of the sections of the tested members were as follows: $D \times t$ = 165.2×4.08 mm, f_{sy} = 353 MPa, f_{ck} = 34.2 MPa. Slenderness ratios (λ) and load eccentricities (e) were shown in the figures. Reasonable agreement is achieved.

3. Parametric analysis and simplified model

3.1. Column strength

3.1.1. Section capacity

For convenience of analysis, "Nominal yielding strength" of the composite sections (f_{scy}) is defined

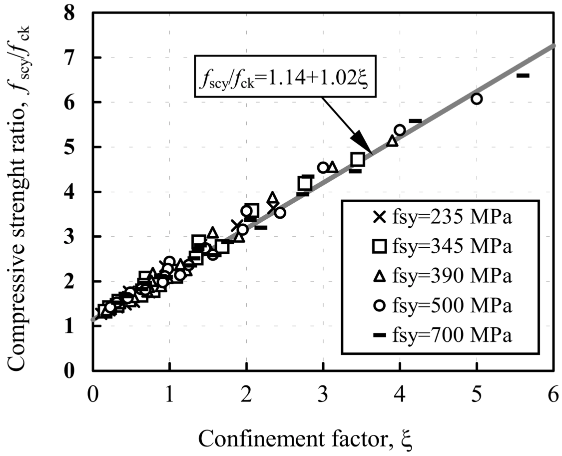


Fig. 6 f_{scy} / f_{ck} versus ξ relations

as follows:

$$f_{scy} = \frac{N_{uo}}{A_{sc}} \tag{9}$$

in which

N_{uo} - Sectional capacity of the composite columns;

A_{sc} - Cross-section area of the compsite sections, given by $\pi \cdot D^2 / 4$.

The compressive strength ratio of the composite sections (f_{scy} / f_{ck}) so determined according to the mechanics model in this paper is plotted in Fig. 6 against the confinement factor (ξ). It can be seen from Fig. 6 that the ratio of f_{scy} / f_{ck} increases when the confinement factor (ξ) increases.

A formula for f_{scy} can be obtained by using regression analysis method, i.e.,

$$f_{scy} = (1.14 + 1.02\xi) \cdot f_{ck} \tag{10}$$

From Eqs. (9) and (10), the section capacity of the composite columns can be expressed as:

$$N_{uo} = A_{sc} \cdot f_{scy} = A_{sc} \cdot (1.14 + 1.02\xi) \cdot f_{ck} \tag{11}$$

3.1.2. Member capacity

Stability reduction factor (φ) for the slender composite columns is defined as follows, i.e.,

$$\varphi = \frac{N_u}{N_{uo}} \tag{12}$$

in which, N_u is the member capacity of the composite columns, N_{uo} is the section capacity, shown as in Eq. (11).

It was found that the stability reduction factor (φ) mainly depends on the slenderness ratio (λ), the strength of steel (f_{sy}), the compression strength of concrete (f_{ck}), and the steel ratio ($\alpha = A_s / A_c$).

Fig. 7 shows the stability reduction factor (φ) versus the slenderness ratio (λ) of the composite columns under difference parameters.

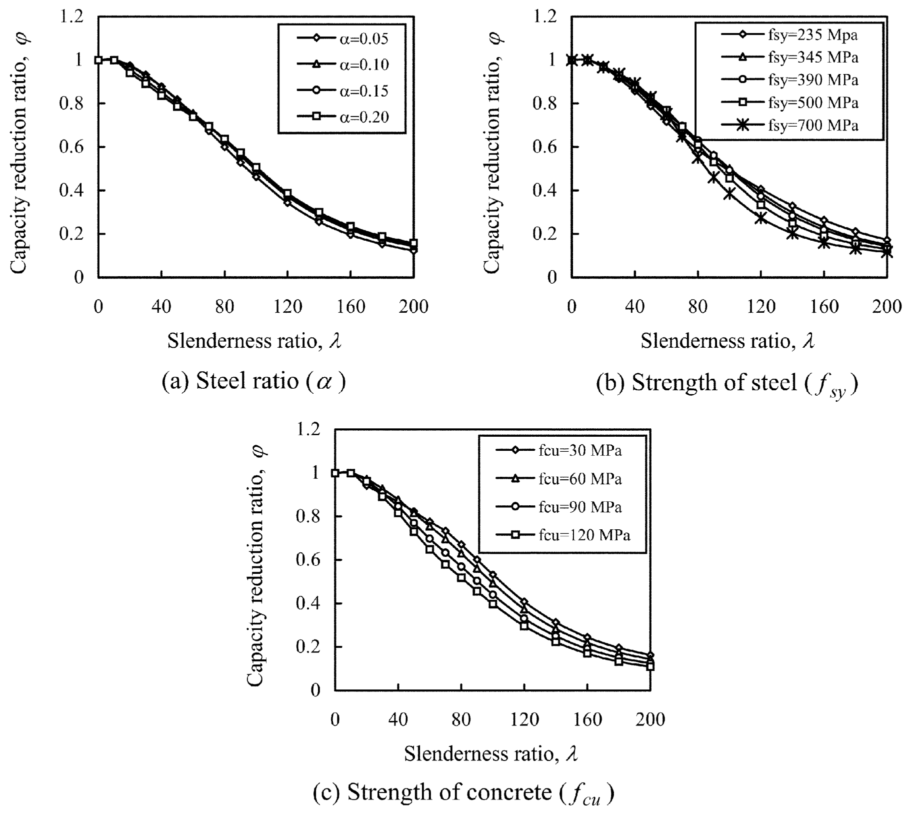


Fig. 7 Capacity reduction ratio (φ) versus slenderness ratio (λ) curves

Using the relations between the stability reduction factor (φ) and other parameters, i.e. the slenderness ratio (λ), the strength of steel (f_{sy}), the strength of concrete (f_{ck}), and the steel ratio ($\alpha = A_s / A_c$), the following formula for φ can be obtained by using regression analysis method, i.e.

$$\varphi = \begin{cases} 1 & (\lambda \le \lambda_o) \\ a \cdot \lambda^2 + b \cdot \lambda + c & (\lambda_o < \lambda \le \lambda_p) \\ \dfrac{d}{(\lambda + 35)^2} & (\lambda > \lambda_p) \end{cases} \tag{13}$$

where

$$a = \frac{1 + (35 + 2 \cdot \lambda_p - \lambda_o) \cdot e}{(\lambda_p - \lambda_o)^2}$$

$$b = e - 2 \cdot a \cdot \lambda_p$$

$$c = 1 - a \cdot \lambda_o^2 - b \cdot \lambda_o$$

$$d = \left[13000 + 4657 \cdot \ln\left(\frac{235}{f_y}\right)\right] \cdot \left(\frac{25}{f_{ck} + 5}\right)^{0.3} \cdot \left(\frac{\alpha}{0.1}\right)^{0.05}$$

$$e = \frac{-d}{(\lambda_p + 35)^3}$$

$$\lambda_o = \pi \cdot \sqrt{\frac{420 \cdot \xi + 550}{f_{scy}}}$$

$$\lambda_p = \pi \cdot \sqrt{\frac{E_s}{0.67 \cdot f_{sy}}}$$

In which E_s is the Young's modulus of steel; f_{scy} is given in Eq. (10).

From Eqs. (11) and (12), the member capacity of the composite columns can be expressed as:

$$N_u = \varphi \cdot N_{uo} = \varphi \cdot f_{scy} \cdot A_{sc} \tag{14}$$

3.2. Beams

For convenience of analysis, flexural strength index (γ_m) is defined in this paper. It is expressed as:

$$\gamma_m = \frac{M_u}{W_{scm} \cdot f_{scy}} \tag{15}$$

in which

M_u - Moment capacity of the composite members;

W_{scm} - Section modulus of the compsite sections, given by $(\pi \cdot D^3)/32$;

f_{scy} - "Nominal yielding strength" of the composite sections, and be given by Eq. (10).

The flexural strength index (γ_m) so determined according to the mechanics model described above is plotted in Fig. 8 against the confinement factor (ξ). It can be seen from Fig. 8 that γ_m increases when the confinement factor (ξ) increases. A formula for the flexural strength index (γ_m) can be obtained by using regression analysis method, i.e.

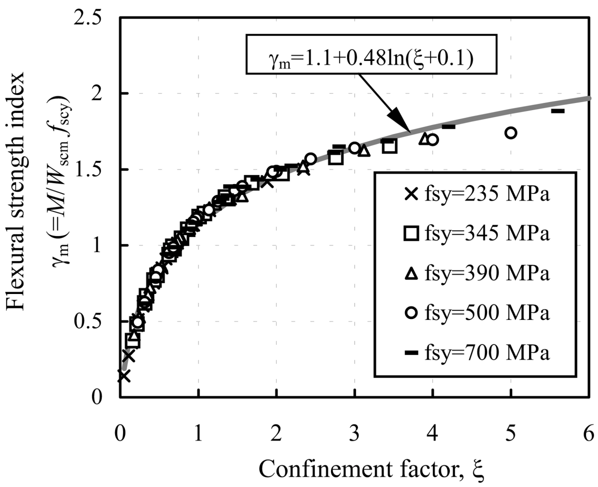


Fig. 8 γ_m versus ξ relations

$$r_m = 1.1 + 0.48 \cdot \ln(\xi + 0.1) \tag{16}$$

According to Eq. (15), the flexural capacity of the composite beam (M_u) can be given by

$$M_u = \gamma_m \cdot W_{scm} \cdot f_{scy} \tag{17}$$

3.3. Beam-columns

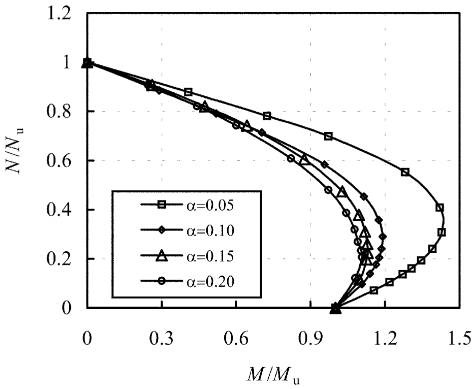
It was found that the interaction relationship between compression strength and bending strength relationship mainly depends on the strength of steel (f_{sy}), the compression strength of concrete (f_{ck}), the steel ratio ($\alpha = A_s / A_c$), and the slenderness ratio (λ).

For convenience of analysis, axial load ratio (η) and bending moment ratio (ς) are defined in this paper, i.e.

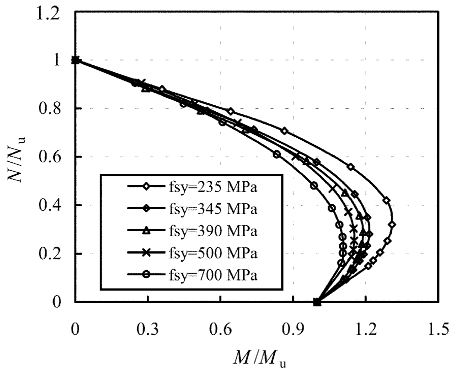
$$\eta = \frac{N}{N_u} \tag{18}$$

$$\varsigma = \frac{M}{M_u} \tag{19}$$

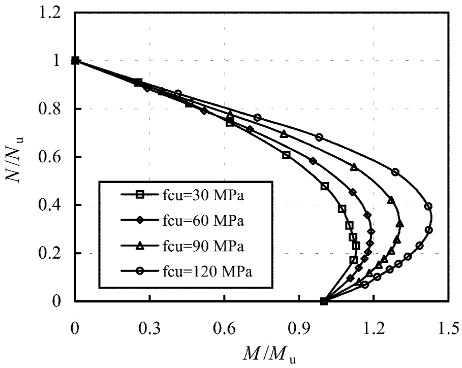
Fig. 9 shows the η versus ς relations of the composite beam-columns under different α, f_{sy} and f_{cu}.



(a) Steel ratio (α)

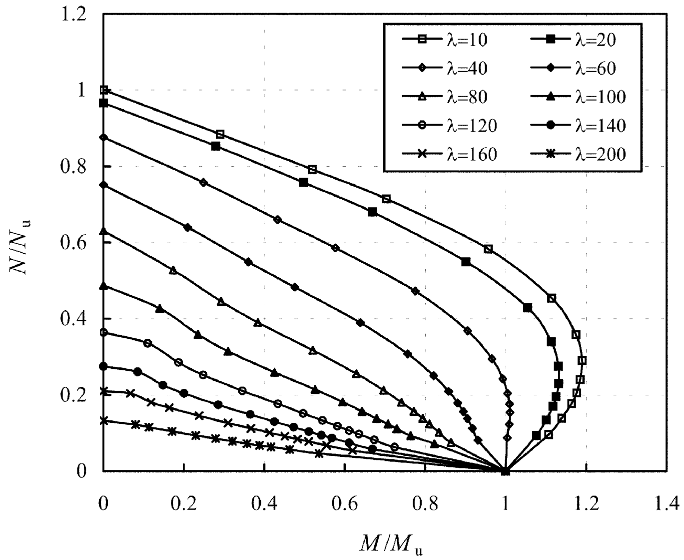


(b) Strength of steel (f_{sy})



(c) Strength of concrete (f_{cu})

Fig. 9 *N/Nu* versus *M/Mu* curves



(D = 400mm ; α = 0.1 ; f_{sy} = 390MPa ; f_{cu} = 60MPa)

Fig. 10 *N/Nu* versus *M/Mu* relations

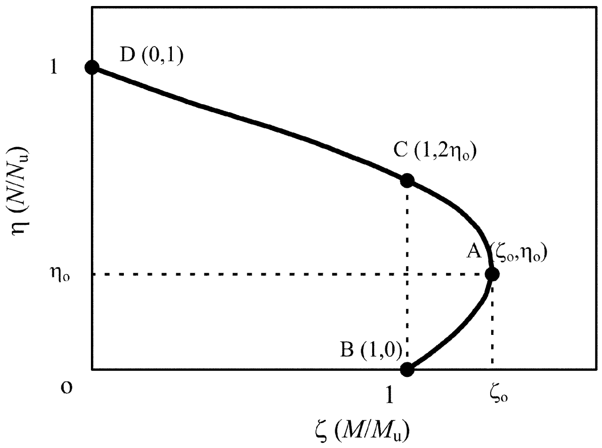


Fig. 11 A schematic view of an interaction curve

Fig. 10 shows the η versus ς relations of the composite members under different slenderness ratio (λ).

Simplified models can be established based on regression analysis for interaction relationship between compression strength and bending strength. A schematic view of an interaction curve for the composite beam-columns is shown in Fig. 11. The coordinates of the contraflexure point A (ς_o , η_o) in Fig. 11 can be calculated as:

$$\varsigma_o = 0.18\xi^{-1.15} + 1 \tag{20}$$

$$\eta_o = \begin{cases} 0.5 - 0.2445 \cdot \xi & (\xi \le 0.4) \\ 0.1 + 0.14 \cdot \xi^{-0.84} & (\xi > 0.4) \end{cases} \tag{21}$$

The η versus ς relations of the composite beam-columns can be expressed as:

$$\frac{1}{\varphi} \cdot \eta + \frac{a}{d} \cdot \varsigma = 1 \qquad (\text{for } \eta \geq 2\varphi^3 \cdot \eta_o) \tag{22a}$$

$$-b \cdot \eta^2 - c \cdot \eta + \frac{1}{d} \cdot \varsigma = 1 \qquad (\text{for } \eta < 2\varphi^3 \cdot \eta_o) \tag{22b}$$

$$a = 1 - 2\varphi^2 \cdot \eta_o$$

$$b = \frac{1 - \varsigma_o}{\varphi^3 \cdot \eta_o^2}$$

$$c = \frac{2 \cdot (\varsigma_o - 1)}{\eta_o}$$

$$d = 1 - 0.4 \cdot \left(\frac{N}{N_E}\right)$$

$$N_E = \pi^2 \cdot E_{sc}^{elastic} \cdot A_{sc} / \lambda^2$$

$$E_{sc}^{elastic} = \frac{f_{scp}}{\varepsilon_{scp}}$$

$$f_{scp} = \left(0.192 \cdot \frac{f_{sy}}{235} + 0.448\right) \cdot f_{scy};\ \varepsilon_{scp} = 0.67 \cdot \frac{f_{sy}}{E_s}$$

The validity limits of Eq. (22) are: D = 100 mm to 2000 mm; α = 0.04 to 0.2; λ = 10 to 200; f_{sy} = 200 MPa to 700 MPa and f_{ck} = 20 MPa to 80 MPa. It should be noted that in Eq. (22), the tube diameter range being 2000 mm was based on numerical simulation due to the difficulties in testing such a large size specimen.

4. Comparison with current code provisions

The section capacity, the member capacity and the moment capacity of concrete-filled steel CHS members predicted using the following five design methods are compared with the test results obtained from different researchers:

(1) LRFD (AISC 1999)
(2) AIJ (1997)
(3) BS5400 (1979)
(4) EC 4 (1994)
(5) The proposed method in this paper.

In all design calculations, the material partial safety factors were set to unity.

4.1. Comparison of section capacity

The section capacities predicted using the five design methods mentioned above are compared with 95 stub column test results obtained from Gardner and Jacobson (1967), Luksha and Nesterovich

Table 1 Comparison between predicted section capacities and test results (stub columns)

No.	D/t	f_{sv} (MPa)	f_{cu} (MPa)	N_{uc}/N_{ue} Formula (11) Mean	COV	LRFD (1999) Mean	COV	AIJ (1997) Mean	COV	EC4 (1994) Mean	COV	BS5400 (1979) Mean	COV	Number of tests	Teat data resource
1	29.5-48.5	363-633	27-52	0.879	0.091	0.706	0.069	0.709	0.069	0.967	0.116	1.196	0.158	14	Gardner and Jacobson (1967)
2	31.4-105.8	291-382	20-55	0.930	0.033	0.755	0.032	0.760	0.033	0.899	0.021	1.066	0.037	10	Luksha and Nesterovich (1991)
3	58.5-220.9	186-363	57-88	0.988	0.125	0.850	0.102	0.881	0.097	0.878	0.117	0.954	0.158	12	O'shea and Bridge (1997a)
4	58.5-220.9	186-363	55-124	0.966	0.054	0.852	0.050	0.889	0.051	0.843	0.049	0.870	0.083	28	O'shea and Bridge (1997b)
5	92.1	270-328	83-95	0.921	0.056	0.822	0.038	0.871	0.039	0.812	0.052	0.878	0.050	6	Prion and Boehme (1994)
6	19.8-58	248-283	28-55	0.974	0.052	0.789	0.039	0.791	0.039	1.038	0.110	1.228	0.156	12	Sakino and Hayashi (1991)
7	17-250	244-320	23-45	0.829	0.069	0.684	0.056	0.686	0.057	0.830	0.151	0.949	0.231	10	Sakino *et al*. (1985)
8	21-46.9	285-537	30-35	0.830	0.129	0.686	0.099	0.725	0.102	0.832	0.102	1.060	0.090	3	Schneider (1998)
Total range	17-250	186-633	20-124	0.947	0.094	0.794	0.088	0.799	0.097	0.916	0.118	1.054	0.191	95	

Table 2 Comparison between predicted member capacities and test results (columns)

No.	D/t	λ	f_{sy} (MPa)	f_{cu} (MPa)	N_{uc}/N_{ue} Formula (14) Mean	COV	LRFD (1999) Mean	COV	AIJ (1997) Mean	COV	EC4 (1994) Mean	COV	BS5400 (1979) Mean	COV	Number of tests	Teat data resource
1	36-98.3	24-32	290-414	27-43	1.061	0.057	0.887	0.06	0.948	0.076	0.923	0.045	1.141	0.071	8	Furlong (1967)
2	29.5-48.5	31.9-79.7	363-605	27-50	0.984	0.145	0.797	0.097	0.841	0.237	0.802	0.089	0.989	0.160	10	Gardner and Jacobson (1967)
3	24	130-154	348	32-47	0.871	0.051	0.846	0.053	0.639	0.036	0.703	0.082	0.811	0.059	11	Han (2000a)
4	36.9-40.3	16-130.2	340-353	50-73	1.018	0.058	0.956	0.088	0.788	0.220	0.921	0.057	1.045	0.086	27	Kato (1996)
5	7.4-50.6	34.7-83.4	235-352	26-37	0.797	0.068	0.715	0.074	0.731	0.148	0.728	0.080	0.824	0.121	21	Kloppel and Goder (1957)
6	15.2-59.1	22.9-83.7	400-482	47	0.912	0.101	0.820	0.112	0.779	0.139	0.848	0.095	1.001	0.101	11	Knowles and Park (1969)
7	34.7-41.8	24-72.4	461-505	57-65	1.026	0.040	0.905	0.056	0.794	0.136	0.909	0.042	1.073	0.094	10	Masuo *et al*. (1991)
8	40.5	16-120	353	50	1.018	0.072	0.910	0.029	0.773	0.191	0.920	0.077	1.043	0.172	6	Matsui *et al*. (1995)
9	7.5-98.3	15.7-168	275-682	26-76	0.903	0.126	0.800	0.094	0.802	0.167	0.815	0.092	0.961	0.163	63	Task Group 20 (1997)
Total range	7.4-98.3	15.7-168	235-682	26-76	0.932	0.119	0.832	0.111	0.787	0.174	0.838	0.099	0.971	0.155	167	

(1991), O'shea and Bridge (1997a), O'shea and Bridge (1997b), Prion and Boehme (1994), Sakino and Hayashi (1991), Sakino *et al*. (1985) and Schneider (1998) in Table 1.

Results in this table clearly show that both LRFD (1999), AIJ (1997) and EC4 (1994) are conservative for predictions of the sectional capacities of the stub columns. Overall, LRFD (1999) and AIJ (1997) gave a section capacity about 20% lower than tested values. EC4 (1994) gave a section capacity about 8% lower than that from tests. BS5400 (1979) predicted slightly higher (within 5%) capacity than the test results. The proposed method gave a capacity about 5% on average lower than that obtained in the tests.

4.2. Comparison of column member capacity

Table 2 shows the comparisons between column member capacity predicted using the five design methods and 167 column test results obtained from Furlong (1967), Gardner and Jacobson (1967), Han (2000a), Kato (1996), Kloppel and Goder (1957), Knowles and Park (1969), Masuo *et al*. (1991), Matsui *et al*. (1995) and Task Group 20 (1997).

Results in this table clearly show that LRFD (1999), AIJ (1997) and EC4 (1994) are conservative for predictions of the column member capacities. Overall, LRFD (1999), AIJ (1997) and EC4 (1994) gave a section capacity about 17-21% lower than experimental values. BS5400 (1979) predicted slightly (3%) lower capacity than the test results. The proposed method predicted about 7% lower capacity than the test results.

4.3. Comparison of beam moment capacity

The moment capacity predicted using the five design methods are compared with 16 beam test results obtained from Elchalakani *et al*. (2001), Prion and Boehme (1994) in Table 3.

Predicted section capacities (M_{uc}) using the different methods are compared with the 16 beam test results in Table 3. Table 3 shows both the mean value and the standard deviation (COV) of the ratio of M_{uc} / M_{ue} for the different design methods. Results in this table clearly show that all of the methods are conservative. Overall, AIJ (1997) and LRFD (1999) gave a moment capacity about 25% lower than those of test results. BS5400 (1979) gave a moment capacity about 18% lower than tested values. EC4 (1994) and the proposed method in this paper give a mean value of 0.865 and 0.892, a COV of 0.124 and 0.119 respectively.

4.4. Comparison of beam-column member capacity

The member capacities of a total of 106 beam-columns obtained from Han (2000b), Johansson and Gylltoft (2001), Kilpatrick and Rangan (1997), Matsui *et al*. (1995), Neogi *et al*. (1969), O'shea and Bridge (1997a), Rangan and Joyce (1991), are compared with member capacity predicted using the five design methods in Table 4.

Predicted section capacities (N_{uc}) using the different methods are compared with 106 experimental results (N_{ue}). Table 4 shows both the mean value and the standard deviation (COV) of the ratio of N_{uc} / N_{ue} for the different design methods. Results in this table clearly show that the LRFD (1999), AIJ (1997) and BS5400 (1979) are conservative. Overall, LRFD (1999), AIJ (1997) and BS5400 (1979) gave a member capacity about 13-28% lower than that of test. EC4 (1994) and the proposed method predicted a slightly (within 4%) lower capacity than the test results.

Table 3 Comparison between predicted moment capacities and test results (beams)

No.	*D/t*	f_{sy} (MPa)	f_{cu} (MPa)	N_{uc} / N_{ue} Formula (17)		LRFD (1999)		AIJ (1997)		EC4 (1994)		BS5400 (1979)		Number of tests	Teat data resource
				Mean	COV	Mean	COV	Mean	COV	Mean	COV	Mean	COV		
1	17-109.9	419	39	0.929	0.117	0.827	0.133	0.835	0.129	0.908	0.106	0.890	0.124	12	Elchalakani *et al.* (2001)
2	92.1	262	83	0.734	0.047	0.505	0.033	0.518	0.033	0.714	0.046	0.632	0.041	4	Prion and Boehme (1994)
Total range	17-109.9	262-419	39-83	0.892	0.119	0.746	0.184	0.756	0.180	0.865	0.124	0.825	0.158	16	

Table 4 Comparison between predicted member capacities and test results (beam-columns)

No.	*D/t*	λ	*e/r*	f_{sy} (MPa)	f_{cu} (MPa)	N_{uc} / N_{ue} Formula (22)		LRFD (1999)		AIJ (1997)		EC4 (1994)		BS5400 (1979)		Number of tests	Teat data resource
						Mean	COV	Mean	COV	Mean	COV	Mean	COV	Mean	COV		
1	29.6-35	12-73.7	0.21-0.57	303-324	44-52	0.784	0.050	0.565	0.092	0.0651	0.046	0.765	0.088	0.778	0.020	7	Han (2000b)
2	92.4	28.11	0.14-0.20	380	101	0.874	0.055	0.521	0.045	1.067	0.043	0.898	0.058	0.861	0.057	3	Johansson and Gylltoft (2001)
3	32.5-42.4	42.2-126.4	0.20-0.98	320-325	66-67	0.910	0.075	0.823	0.133	0.661	0.049	1.026	0.066	0.865	0.098	16	Kilpatrick and Rangan (1997)
4	40.5	16-120	0.13-0.25	353-433	50-79	1.029	0.077	0.888	0.180	0.899	0.266	1.080	0.122	0.982	0.089	21	Matsui *et al.* (1995)
5	14.4-78.4	44.4-95.1	0.1-0.56	193-312	27-83	0.933	0.204	0.807	0.225	0.678	0.131	1.035	0.210	0.881	0.142	18	Neogi *et al.* (1969)
6	58.5-220.9	14	0.07-0.22	186-363	55-124	1.051	0.081	0.457	0.145	1.007	0.145	0.894	0.070	0.740	0.241	23	O'shea and Bridge (1997a)
7	23.8-63.5	31.8-92.2	0.20-0.591	218-341	77-95	0.952	0.085	0.759	0.189	0.738	0.198	1.065	0.110	0.936	0.110	18	Rangan and Joyce (1991)
Total range	14.4-220.9	12-126.4	0.07-0.98	186-433	27-124	0.968	0.126	0.718	0.232	0.812	0.218	0.998	0.151	0.870	0.166	106	

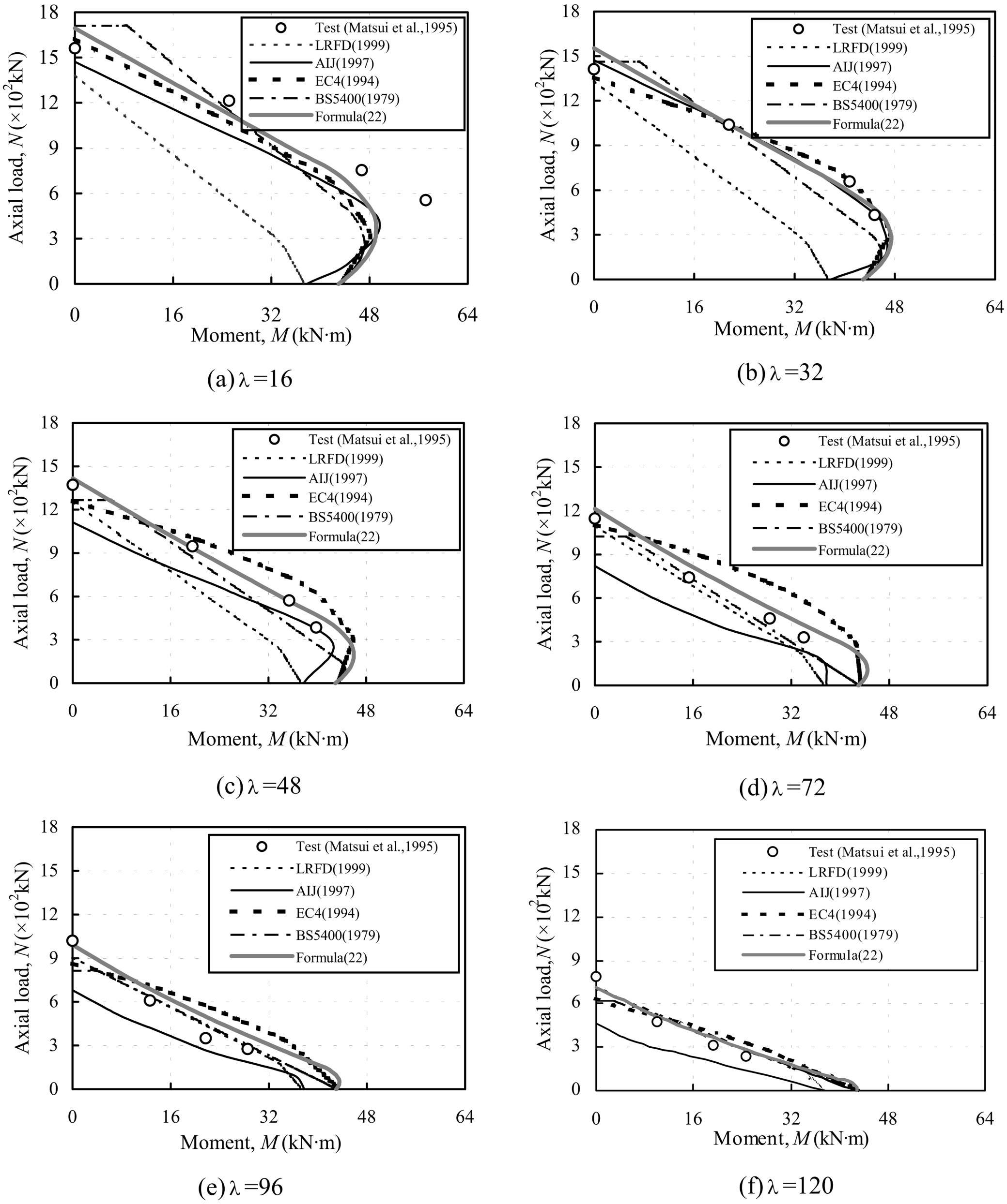


Fig. 12 Comparison of predicted interaction curves with test results by Matsui *et al.* (1995)

The predicted axial load (N) versus moment (M) interaction curves using different methods are compared in Fig. 12 with a set of experimental values obtained by Matsui *et al.* (1995), parameters of the sections of the tested members were as follows: $D \times t = 165.2 \times 4.08$ mm, $f_{sy} = 353$ MPa, $f_{ck} = 34.2$ MPa. Slenderness ratios are shown in the figures. Reasonable agreement was achieved between Eq. (22) and test results.

5. Conclusions

The following observations and conclusions are made based on the limited research reported in the paper.

(1) A unified theory is described where a confinement factor (ξ) is introduced to describe the composite action between the steel tube and filled concrete in this paper. A mechanics model is developed for concrete-filled steel CHS columns and beam-columns. The predicted column strength is compared with 384 tests results; reasonable agreement was obtained with the ratio of predicted to experimental capacity ranges from 0.86 to 0.97.

(2) The load versus mid-span deflection relationship is established for concrete-filled steel CHS columns and beam columns theoretically. The predicted curves of load versus mid-span deflection are generally in good agreement with test results.

(3) A simplified model is developed for calculating the section capacity, member capacity and moment capacity of concrete-filled steel CHS members. Simplified interaction curves are derived for concrete-filled steel CHS beam-columns. Comparisons are also made with predicted column strengths using LRFD (1999), AIJ (1997), BS5400 (1979) and EC4 (1994). The codes are found (about 10% to 25%) conservative in general. The capacities predicted by the simplified model is about 4% to 10% lower than those obtained in the tests.

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Notation

A_s	: Steel cross-sectional area
A_c	: Concrete cross-sectional area
CHS	: Circular hollow section
D	: Diameter of circular steel tube
e	: Eccentricity of load
e/r	: Load eccentricity ratio, $r = D/2$
E_c	: Concrete modulus of elasticity
E_s	: Steel modulus of elasticity
f_{sy}	: Yield strength of steel
f_{cu}	: Characteristic 28-day concrete cube strength
f_{ck}	: Characteristic concrete strength ($f_{ck} = 0.67\ f_{cu}$ for normal strength concrete)
I_{sc}	: Moment of inertia for composite cross section, given by $\pi \cdot D^4/64$
L	: Effective buckling length of column in the plane of bending, in mm
M	: Moment
M_u	: Moment capacity
M_{uc}	: Predicted moment capacity
M_{ue}	: Maximum test moment
N	: Axial load
N_u	: Axially compressive capacity
N_{uc}	: Predicted ultimate strength
N_{ue}	: Experimental ultimate strength
t	: Wall thickness of steel tube
u_m	: Mid-height deflection of the column
W_{scm}	: Section modulus of the composite sections, given by $\pi \cdot D^3/32$
α	: Steel ratio (= A_s / A_c)
σ	: Stress
ε	: Strain
λ	: Slenderness ratio, given by $4L/D$
ξ	: Confinement factor ($\xi = \dfrac{A_s \cdot f_{sy}}{A_c \cdot f_{ck}}$)
γ_m	: Flexural strength index

Appendix I Stress-strain relationship for steel

$$\sigma = E_s \cdot \varepsilon \qquad \text{for} \quad \varepsilon \le \varepsilon_1 \tag{I.1a}$$

$$\sigma = -A \cdot \varepsilon^2 + B \cdot \varepsilon + C \qquad \text{for} \quad \varepsilon_1 < \varepsilon \le \varepsilon_2 \tag{I.1b}$$

$$\sigma = f_{sy} \qquad \text{for} \quad \varepsilon_2 < \varepsilon \le \varepsilon_3 \tag{I.1c}$$

$$\sigma = f_{sy} \cdot \left[1 + 0.6 \cdot \frac{\varepsilon - \varepsilon_3}{\varepsilon_4 - \varepsilon_3}\right] \qquad \text{for} \quad \varepsilon_3 < \varepsilon \le \varepsilon_4 \tag{I.1d}$$

$$\sigma = 1.6 \cdot f_{sy} \qquad \text{for} \quad \varepsilon > \varepsilon_4 \tag{I.1e}$$

where, E_s is the steel modulus of elasticity, and is taken as 200,000 MPa in this paper; $\varepsilon_1 = 0.8 \cdot f_{sy} / E_s$; $\varepsilon_2 = 1.5 \cdot \varepsilon_1$; $\varepsilon_3 = 10 \cdot \varepsilon_2$; $\varepsilon_4 = 100 \cdot \varepsilon_2$. f_{sy} is the yielding strength of the steel.

Appendix Ⅱ Stress-strain relationship for confined concrete

$$y = 2x - x^2 \qquad (x \le 1) \tag{II.1a}$$

$$y = \begin{cases} 1 + q \cdot (x^{0.1\xi} - 1) & (\xi \ge 1.12) \\ \dfrac{x}{\beta \cdot (x-1)^2 + x} & (\xi < 1.12) \end{cases} \qquad (x > 1) \tag{II.1b}$$

in which, $y = \sigma / \sigma_o$, $x = \varepsilon / \varepsilon_o$

$$\sigma_o = f_{ck} \cdot \left[1.194 + \left(\frac{13}{f_{ck}}\right)^{0.45} \cdot (-0.07845 \cdot \xi^2 + 0.5789 \cdot \xi)\right] \tag{II.2}$$

$$\varepsilon_o = \varepsilon_{cc} + 0.95 \cdot \left[1400 + 800 \cdot \left(\frac{f_{ck} - 20}{20}\right)\right] \cdot \xi^{0.2} \tag{II.3}$$

where, $\varepsilon_{cc} = (1300 + 14.93\, f_{ck})$

$$k = 0.1 \cdot \xi^{\,0.745}$$

$$q = \frac{k}{0.2 + 0.1 \cdot \xi}$$

$$\beta = (2.36 \times 10^{-5})^{[0.25 + (\xi - 0.5)^7]} \cdot f_{ck}^2 \cdot (5 \times 10^{-4})$$

The units for stress and strain are MPa and $\mu\varepsilon$ respectively.

CC