State of the art report on steel–concrete composite columns

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Abstract

Steel–concrete composite columns are used extensively in modern buildings. Extensive research on composite columns in which structural steel section are encased in concrete have been carried out. In-filled composite columns, however have received limited attention compared to encased columns. In this paper, a review of the research carried out on composite columns is given with emphasis on experimental and analytical work. Experimental data has been collected and compiled in a comprehensive format listing parameters involved in the study. The review also includes research work that has been carried out to date accounting for the effects of local buckling, bond strength, seismic loading, confinement of concrete and secondary stresses on the behaviour of steel–concrete composite columns. © 2001 Elsevier Science Ltd. All rights reserved.

Keywords: Composite columns; Concrete filled; Concrete encased

1. Introduction

Two types of composite columns, those with steel section encased in concrete and those with steel section in-filled with concrete are commonly used in buildings. Basic

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forms of cross-sections representative of composite columns are indicated in Fig. 1. Concrete-encased steel composite columns have become the preferred form for many seismic-resistant structures. Under severe flexural overload, concrete encasement cracks resulting in reduction of stiffness but the steel core provides shear capacity and ductile resistance to subsequent cycles of overload.

Concrete-filled steel tubular columns have been used for earthquake-resistant structures, bridge piers subject to impact from traffic, columns to support storage tanks, decks of railways, columns in high-rise buildings and as piles. Concrete-filled steel tubes require additional fire-resistant insulation if fire protection of the structure is necessary. Because of the increased use of composite columns, a great deal of theoretical and experimental work has been carried out.

This paper presents the state of art knowledge on steel–concrete composite columns including experimental and analytical studies. A summary of experiments reported in literature is presented in a tabular form. The discussion includes behaviour of short and slender composite columns. Use of high strength concrete in composite columns is briefly outlined. A detailed discussion on the effect of local buckling, bond strength, confinement of concrete, seismic behaviour and secondary stresses on composite columns are presented.

2. Short columns

In the early stages of loading, Poisson's ratio for concrete is lower than that for steel, and the steel tube has no restraining effect on the concrete core. As the longitudinal strain increases, Poisson's ratio of concrete which is 0.15–0.2 in the elastic range increases to 0.5 in the inelastic range [1]. Therefore, the lateral expansion of uncontained concrete gradually becomes greater than that of steel. A radial pressure develops at the steel–concrete interface thereby restraining the concrete core and setting up a hoop tension in the tube. At this stage, the concrete core is stressed triaxially and the steel tube biaxially, so that there is a transfer of load from the tube
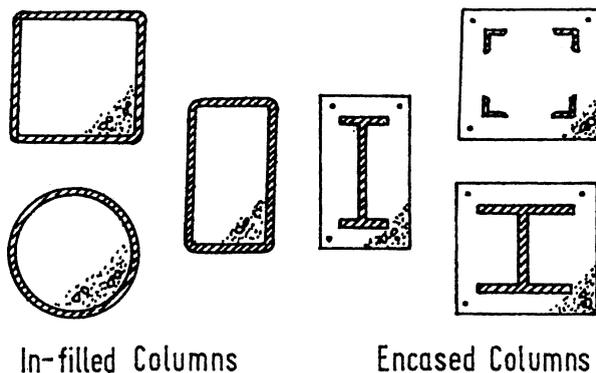

In-filled Columns   Encased Columns

Fig. 1.   Types of composite column [1].

to the core, as the tube cannot sustain the yield stress longitudinally in the presence of a hoop tension. The load corresponding to this mode of failure can be considerably greater than the sum of the steel and concrete, but shear failure may intervene before the load transfer is complete [2].

2.1. Interaction diagram

When a short composite column is subjected to a small axial load, the composite section is capable of supporting a bending moment in excess of its ultimate moment of resistance, M_u. This is similar to the effect of a prestressing force on a reinforced concrete section. If the axial force and bending moment are independently applied to the composite section, it should be noted that the removal of the axial load would destabilize the section as the corresponding point on the interaction diagram would fall outside the failure envelope. It is, therefore, recommended to consider the cut off point at $M/M_u = 1.0$ to form an integral part of the interaction curve [3].

2.2. Axially loaded member

The plastic resistance of the cross-section of a composite column with concentric loading is given by:

$$N_{\text{pl.rd}} = A^a f_{yd} + A^c f_{cd} + A^s_z f_{sd} \tag{1}$$

in which A^a, A^c and A^s_z are cross-sectional area of structural steel, concrete and reinforcement respectively in the axial direction; f_{yd}, f_{cd} and f_{sd} are, respectively, design strengths of the corresponding materials. Evidence of an increase in concrete strength is obtained for short circular concrete filled steel tubes but not for short columns constructed from square or rectangular hollow steel tubes filled with concrete, probably because the concrete is subjected to a complex three dimensional state of stress.

3. Slender columns

Slender columns are generally subjected to compression and bending. Failure occurs when the conditions of stressing under which stable equilibrium is no longer possible between internal and external forces. At this point, for minimal added strain, the increase in external bending moments is more than that the section can take. The elastic critical buckling stress of an ideally straight column is written as:

$$f_{cr} = \frac{N_{cr}}{A} = \frac{\pi^2 E}{(L/r)^2} \tag{2}$$

This expression is non-dimensionalized as

$$\bar{N} = \frac{f_{cr}}{f_y} = \frac{1}{\lambda^2} \tag{3}$$

where $\lambda = \dfrac{(L/r)}{\lambda_E}$ (4)

and $\lambda_E = \pi\sqrt{\dfrac{E}{f_y}}$ (5)

where:

N_{cr}	the critical load
A	area of cross-section
L	effective length
f_y	yield stress
E	modulus of elasticity and
r	radius of gyration of the column section

λ is called the slenderness factor (to distinguish it from the slenderness ratio). Real columns would not support a stress in excess of f_y, if the effect of strain hardening is ignored. Behaviour of a practical column deviates from that of a similar ideal column, particularly in the range corresponding to $0.3<\lambda<1.5$. The presence of residual stresses within the cross-section, rather than the initial imperfections, may govern the design practice. Some measure of the level of residual stresses within the section can be obtained by conducting a stub column test. The resulting non-linear stress–strain curve can be used to obtain the inelastic critical buckling load of a straight column using the tangent modulus formula proposed by Engesser. Conventional definition of a radius of gyration cannot be applied rigorously to non-homogeneous or composite cross-section. An effective radius of gyration for this composite section will be somewhat greater than the larger of values for each material taken separately. In the case of concrete-filled composite columns, critical load of slender columns are obtained using equivalent radius of gyration and flexural stiffness. Design codes provide different equations for this equivalent stiffness although same design philosophy and data were used as the basis for their formulation.

3.1. P–Δ effect

P–Δ effect is more pronounced in longer columns. Behaviour of such columns could be predicted by the Euler approach, where the effect of initial imperfections is neglected. When the length of the column is increased, the relationship between the applied load and the mid-length moment is no longer linear. The lateral displacements have an adverse effect on the load-carrying capacity of the column as they generate a mid-length secondary moment. As the length of the column is further increased, the secondary bending moment increases significantly and its effect becomes of prime importance. This causes the column to fail by bending rather than by compression and leads to an instability problem. The overall stability of a steel tube filled with concrete will be influenced more by the steel tube than by the contained concrete.

4. Experimental studies

Some of the earlier tests on composite columns were carried out by Burr [4], and were followed over the years by more experimental and theoretical studies by other researchers. Experiments were conducted to obtain basic information to serve as an aid to analyse modeling or to formulate design criteria. The following sections presents summary of experiments on both encased and in-filled composite columns tested by various researchers with a view to establish their behaviour and load-carrying capacity. Experimental works on both encased and in-filled steel–concrete composite columns are listed in Tables 1 and 2, respectively.

4.1. Encased sections

Mirza and Skrabek [5,6] investigated the effect of concrete and steel strengths, the cross-sectional dimensions of concrete and steel section, the presence of steel section and reinforcing bars on strength of composite column. Two slenderness ratios of 0 and 21.9 were examined. The other parameter, eccentricity ratio was varied from zero to 4 at an interval of 0.05. The results indicated that concrete strength, structural steel ratio, and the end eccentricity ratio influence the statistical properties in reliability study of short composite beam-columns. The end eccentricity ratio of 0.5 or less are critical since they have a very significant effect on the beam-column strength.

Hunaiti and Fattah [7] investigated the load-carrying capacity of partially encased composite columns subjected to minor axis bending. IPE 200×100×22 steel sections with a buckling length of 2.4 m were encased partially in concrete and tested to failure under eccentric axial load. The variables studied include eccentricity of the applied load, eccentricity ratio at the column ends, and the effect of concrete strength. In all tests, tension cracks were observed at loads beyond 70% of the failure load. The maximum strength of the columns was obtained by deflection methods using Newmark's technique of numerical integration.

4.2. In-filled sections

Tests were conducted on two 47 ft (14 m) long, 13 in (33 cm) diameter pipe columns, one empty and the other filled with concrete [8]. It was concluded that concrete increases the load and moment carrying capacity without increasing the size of the column. Experiments on columns with steel tubes of diameter-to-thickness ratio (D/t) of 92 and yield stress of f_y in the range of 250–350 MPa, filled with concrete of characteristic strength f_c between 70 and 90 MPa were carried out by Prion and Boehme [9]. The effect of confinement of concrete was noticeable in columns of slenderness ratio L/D less than 15. No appreciable difference in load carrying capacity was detected between long and short specimens and between loading on the whole section and on concrete alone.

Rectangular in-filled composite columns of 3 m long were tested under three different loading viz. axial, uniaxial bending applied about major or minor axis and

Table 1
Experiments on encased composite columns

Sl. no.	Author(s)	Country	Ref. no.	Year	Shape of section	Type of loading	Analysis-A/Expt.-E/Design-D	No. of tests	Variables studied	Remarks
1	Stevens RF	UK	[24]	1965	Square & Rect.	Eccentric	E	11	Eccentricity of applied load, strength of concrete and steel, dimension of section.	Specimens were 4.57 m long and loaded eccentrically along the weak axis
2	Virdi KS, Dowling PJ	UK	[21]	1973	Rect.	Eccentric	A, E	9	Length, eccentricity along major and minor axes, concrete strength	Proposed a numerical analytical method
3	Anslijn R, Janss J	Belgium	[51]	1974	Rect.	Eccentric	E	30	Effective length, relative slenderness, strength of concrete and steel, dimension of section	Specimens were tested without longitudinal reinforcement
4	Roderick JW, Loke YO	Australia	[52]	1975	Rect.	Eccentric	A, D, E	15	Eccentricity of applied load, slenderness ratio.	Proposed a strength calculation method and design procedure

Table 1
(*Continued*)

Sl. no.	Author(s)	Country	Ref. no.	Year	Shape of section	Type of loading	Analysis-A/Expt.-E/Design-D	No. of tests	Variables studied	Remarks
5	Johnston BG	USA	[53]	1976	Square	Eccentric	E	13	Structural steel ratio, eccentricity	Section was subjected to uniaxial moments about both axes and axial load
6	Morino S, Matsui C, Watanabe H	Japan	[13]	1984	Square& Rect.	Eccentric	E	40	Eccentricity of applied load, strength of concrete and steel, dimension of section	Compared with the conventional theoretical analytical method
7	Matsui C	Japan	[54]	1979	Square	Eccentric	E	4	Effective length, relative slenderness, strength of concrete reinforcement and steel	Specimens were tested with longitudinal reinforcement
8	Roik R, Schwalbenhofer K	Germany	[55]	1989	Square	Eccentric	E	27	Eccentricity of applied load, strength of concrete and steel, dimension of section.	Pin-ended columns of length 3 m were tested in uniaxial or biaxial bending with axial load
9	Roik K, Bergmann R	Germany	[56]	1984	Square	Eccentric	D, E	12	Effective length, strength of concrete and steel	Proposed design method and interaction equation

Table 1
(*Continued*)

Sl. no.	Author(s)	Country	Ref. no.	Year	Shape of section	Type of loading	Analysis-A/Expt.-E/Design-D	No. of tests	Variables studied	Remarks
10	Mirza SA, Skrabek BW	Canada	[5]	1991	Square	Eccentric	A, D, E	16	Concrete strength, structural steel ratio, end eccentricity ratio, slenderness ratio	Compared with proposed design method and ACI standards
11	Elnashai AS, Takanashi K, Elghazouli AY Dowling PJ	UK	[57]	1991	Square	Cyclic, Pseudo Dynamic	A, E	6	Type of loading and percentage of axial load.	Results compared with computer program
12	Mirza SA, Skrabek BW	Canada	[6]	1992	Square	Eccentric	E	16	Strain-hardening, strength of concrete and steel, slenderness ratios	Compared against ACI Building code
13	Matsui C	Japan	[43]	1993	Square	Eccentric	A, D, E	6	Effective length, relative slenderness, strength of concrete	Proposed a strength calculation method

Table 1
(*Continued*)

Sl. no.	Author(s)	Country	Ref. no.	Year	Shape of section	Type of loading	Analysis-A/Expt.-E/Design-D	No. of tests	Variables studied	Remarks
14	Nakamura T	Japan	[58]	1994	Square	Eccentric	E	7	Effective length, relative slenderness, strength of concrete	Relative slenderness varied from 0.15 to 1.0
15	Hunaiti YM, Fattah BA	Amman	[7]	1994	Rect.	Eccentric	A, D, E	19	Eccentricity of applied load, eccentricity ratio, concrete strength	Compared with theoretical load of computerized analysis and design load based on BS 5400
16	Wium JA, Lebet JP	S. Africa, Switzerland	[59]	1994	Square	Push-out and Pull-out	A, D, E	27	Thickness of concrete cover, amount of hoop reinforcement, size of steel section, shrinkage of concrete.	Proposed a simplified design method
17	Ricles JM, Paboojian SD	Bethlehem, California	[60]	1994	Square	Seismic	A, E	8	Degree of concrete confinement, shear resistance mechanism, strength of concrete & steel.	Compared against ACI and AISC-LRFD specifications

Table 1
(*Continued*)

Sl. no.	Author(s)	Country	Ref. no.	Year	Shape of section	Type of loading	Analysis-A/Expt.-E/Design-D	No. of tests	Variables studied	Remarks
18	Boyd PF, Cofer WF, McLean DI	USA	[35]	1995	Circular	Axial, Reversed cyclic	A, E	5	D/t ratio, shear studs, strength of concrete and steel.	Proposed a numerical method
19	Mirza SA, Hyttinen V Hyttinen E	Canada, Finland	[61]	1996	Square	Eccentric	A, E	16	Strength of concrete, reinforcing steel and structural steel, eccentricity	Verified against ACI, Eurocode standards and proposed finite element model
20	Munoz PR, Hsu CTT	New Jersey	[19]	1997	Square	Axial & Eccentric	A, E	4	Eccentricity of applied load, slenderness ratio, strength of concrete and steel.	Proposed a computerized numerical method
21	Wang YC	UK	[62]	1999	Square	Eccentric	D, E	7	Eccentricity and moment ratio along both directions, strength of concrete and steel.	Proposed a design method based on BS 5950

Table 2
Experiments on in-filled composite columns

Sl. no.	Author(s)	Country	Ref. no.	Year	Shape of section	Type of loading	Analysis-/AExpt.-E/Design-D	No. of tests	Variable studied	Remarks
1	Neogi PR, Sen HK, Chapman JC	UK	[2]	1969	Circular	Eccentric	A, E	18	D/t ratio, slenderness ratio, strength of concrete and steel, eccentricity	Proposed a computerised numerical method
2	Knowles RB, Park R	New Zealand	[30]	1969	Circular & Square	Axial & Eccentric	A, E	28	Strength of concrete and steel, slenderness ratio	Proposed an analytical method
3	Drysdale RG, Huggins MW	Canada	[63]	1971	Square	Eccentric	A, E	58	Strength of concrete and steel, loading sequence, duration of loading, eccentricity	Proposed a numerical method

Table 2
(*Continued*)

Sl. no.	Author(s)	Country	Ref. no.	Year	Shape of section	Type of loading	Analysis-/AExpt.-E/Design-D	No. of tests	Variable studied	Remarks
4	Bridge RQ	Australia	[64]	1976	Square	Eccentric	A, D, E	8	*D/t* ratio, slenderness ratio, strength of concrete and steel, eccentricity ratio, inclination of loading axis.	Proposed an analytical method and compared with test results
5	Task Group 20 SSRC	USA	[22]	1979	Circular	Eccentric	D, E	51	Effective length, relative slenderness, strength of concrete and steel, dimensions of tube.	Short section were subjected to uniaxial moments about both axes and axial load
6	Sasaki R	Japan	[65]	1984	Circular	Eccentric	D, E	21	Effective length, relative slenderness, strength of concrete and steel	High strength concrete (53 MPa – 63 MPa) was used as in-fill

Table 2
(*Continued*)

Sl. no.	Author(s)	Country	Ref. no.	Year	Shape of section	Type of loading	Analysis-/AExpt.-E/Design-D	No. of tests	Variable studied	Remarks
7	Prion HGL, Boehme J	Canada	[9]	1989	Circular	Eccentric	A, E	20	Method of loading, length, strength of concrete and steel	Developed an analytical method.
8	Shakir Khalil H, Zeghiche J	UK	[10]	1989	Rect.	Eccentric	A, E	7	Strength of concrete and steel, eccentricity ratio.	Compared with finite element analysis and BS 5400.
9	Shakir Khalil H, Mouli M	UK	[11]	1990	Rect.	Axial & Eccentric	E	9	Column section, eccentricity ratio, effective length, strength of concrete and steel.	Compared against BS 5950 & BS 5400 provisions.

Table 2
(*Continued*)

Sl. no.	Author(s)	Country	Ref. no.	Year	Shape of section	Type of loading	Analysis-/AExpt.-E/Design-D	No. of tests	Variable studied	Remarks
10	Konno H, Kei T, Nagashima T.	Japan	[66]	1990	Square	Eccentric	D, E	19	Thickness of steel tube, strength of concrete and steel, and ratio of axial force.	Confinement effect accounted in proposed strength estimation method
11	Cai SH	Japan	[67]	1991	Circular	Eccentric	A, E	27	Effective length, strength of concrete and steel, eccentricity of applied load	Column were subjected to both single and double curvature bending
12	Rangan BV, Joyce M	Australia	[16]	1992	Circular	Eccentric	D, E	9	Slenderness ratio, eccentricity of axial thrust	Proposed a strength calculation method

Table 2
(*Continued*)

Sl. no.	Author(s)	Country	Ref. no.	Year	Shape of section	Type of loading	Analysis-/AExpt.-E/Design-D	No. of tests	Variable studied	Remarks
13	Ge H, Usami T	Japan	[14]	1992	Square	Eccentric	A, E	6	Width to thickness ratio and stiffener rigidity.	Proposed a strength prediction formula
14	CFT working group	Japan	[68]	1993	Circular	Eccentric	D, E	6	Effective length, relative slenderness, strength of concrete and steel	In-fill Concrete was of strength 41 Mpa
15	O'Brien AD, Rangan BV	Australia	[17]	1993	Circular	Eccentric	D, E	9	Effective length, wall thickness, and strength of steel	Proposed a strength calculation method

Table 2
(*Continued*)

Sl. no.	Author(s)	Country	Ref. no.	Year	Shape of section	Type of loading	Analysis-/AExpt.-E/Design-D	No. of tests	Variable studied	Remarks
16	Fuji R	Japan	[69]	1994	Circular	Eccentric	A, E	33	D/t ratio, strength of concrete and steel, effective length.	Verified with proposed strength calculation method and EC4.
17	Kilpatrick AE	Australia	[70]	1994	Circular	Eccentric	A, E	16	D/t ratio, strength of concrete and steel, effective length.	Verified with proposed strength calculation method and EC4.
18	Shakir-Khalil H, Al-Rawdan A	UK	[12]	1994	Rect.	Axial& Eccentric	A, E	15	Effective length, eccentricity of applied load, and strength of concrete and steel	ABAQUS software package was used to verify results

Table 2
(*Continued*)

Sl. no.	Author(s)	Country	Ref. no.	Year	Shape of section	Type of loading	Analysis-/AExpt.-E/Design-D	No. of tests	Variable studied	Remarks
19	Hayashi N, Inone T, Fukumoto T, Ukada T, Akiyama H	Japan	[71]	1995	Square	Displ. controlled	D, E	10	Width to thickness ratio, axial force ratio, strength of concrete and steel.	Proposed a strength calculation method
20	Kim WJ, Park JM, Oh YS	Korea	[72]	1995	Circular	Axial	D, E	14	Diameter to thickness ratio, loading condition, strength of concrete and steel	Proposed an empirical formulae to predict ultimate capacity.
21	Matsui C, Tsuda K, Ishibashi Y	Japan	[49]	1995	Circular& Square	Axial & Eccentric	A, D, E	24	Buckling length, section depth, eccentricity of applied load, slenderness ratio.	Proposed a design method
22	O'Shea MD, Bridge RQ	Australia	[18]	1995	Circular	Axial & Eccentric	E	10	Diameter to thickness ratio, effective length, strength of concrete and steel.	Compared with EC4 codal provisions

Table 2
(*Continued*)

Sl. no.	Author(s)	Country	Ref. no.	Year	Shape of section	Type of loading	Analysis-/AExpt.-E/Design-D	No. of tests	Variable studied	Remarks
23	Bridge RQ, O'Shea MD, Tyrell J, Grigson P, Gardener P	Australia	[73]	1995	Circular	Axial	E	12	Diameter to thickness ratio, effective length, strength of concrete and steel.	Compared with EC4 codal provisions
24	Kilpattrick AE	Australia	[74]	1996	Circular	Eccentric	A, E	57	Bond condition, effective length, age of concrete, slenderness ratio, eccentricity	Proposed a strength calculation method
25	Wang YC, Moore DB	UK	[29]	1997	Rect.	Eccentric	D, E	10	Eccentricity ratios along major and minor axes	Verified with BS 5950, EC4 and the proposed design procedure.
26	Uy B, Das S	Australia	[75]	1997	Square	Eccentric	E	10	Eccentricity of applied load, effective length.	Compared against numerical model predictions.

Table 2
(*Continued*)

Sl. no.	Author(s)	Country	Ref. no.	Year	Shape of section	Type of loading	Analysis-/AExpt.-E/Design-D	No. of tests	Variable studied	Remarks
27	Thirugnana-sundralingam K, Thayalan P, Patnaikuni I	Australia	[76]	1997	Circular	Static & Variable repeated loading	E	8	Eccentricity of applied load, concrete strength.	Proposed a strength calculation method
28	Kawano A, Matsui C	Japan	[77]	1997	Circular	Cyclic	A, E	44	Slenderness ratio, D/t ratio, strength of concrete and steel.	Proposed an empirical design equation.
29	Uy B	Australia	[47]	1998	Square	Displ. controlled	A, E	10	Dimensions of section, method of loading, residual stress	Compared with proposed analysis method and AS 4100 & BS 5950 codes.
30	Wang YC	UK	[62]	1999	Rect.	Eccentric	D, E	8	Eccentricity and moment ratio along both directions, strength of concrete and steel.	Proposed a design procedure based on BS 5950.
31	Nakanishi K, Nakai H, Kitada T	Japan	[37]	1999	Square	Seismic	E	8	Cross-section, strength of steel and concrete, natural period of vibration.	Proposed an empirical design equation.

biaxial bending [10]. Effect of end eccentricity ratios e_x/D and e_y/D up to 0.5 was also studied. In most columns, the yield strain was reached in the compressive zones at loads varying between 80% and 90% of the failure load. After the failure, the tensile strains reached the yield as the columns underwent large lateral displacements and consequently were subjected to high bending moments. It was concluded that the failure mode of all columns was an overall buckling mode, with no sign of local buckling of the steel section. Further tests on rectangular in-filled steel tubes by Shakir-Khalil and Mouli [11] showed that concrete filling increases considerably the load carrying capacity of the column. In addition to varying the column section, the effects of varying the design strength of steel, the cube strength of concrete and the column length were also investigated. The relative carrying capacity of composite to steel column increases when the size of the steel section is increased and with the use of high strength concrete.

Failure loads and load–displacement relationships up to and beyond failure of concrete-filled composite columns of 3, 4 and 5 m were analysed using the software ABAQUS by Shakir-Khalil and Rawdan [12]. Tensile strength of the concrete was taken as 10% of the compressive strength in bending. i.e. $0.06f_{cu}$. Results of an experimental study on the elasto-plastic behaviour of steel reinforced concrete (SRC) columns subjected to biaxially eccentric compression showed that a sharp peak appears on the load–deflection curve of a short column because of a concrete crush, while P–Δ effect was more pronounced in a long column and a gradual unloading takes place [13].

4.3. Failure modes

Short composite columns exhibit a failure mechanism characterized by yielding of steel and crushing of concrete. Medium length columns behave inelastically and fail by partial yielding of steel, crushing of concrete in compression and cracking of concrete in tension. When the load applied to a column is eccentric in the stronger plane of bending and the slenderness for buckling in that plane is much less than that for minor-axis buckling, failure in a biaxial mode is possible. In columns subjected to biaxial bending, the neutral axis changes its position continuously by a combination of translation and rotation. Stiffness along the whole length of the column varies due to an uncracked concrete section near the ends with an increasing frequency of cracking nearer the center. Ge and Usami [14] studied local buckling modes of stiffened and unstiffened in-filled columns shown in Fig. 2.
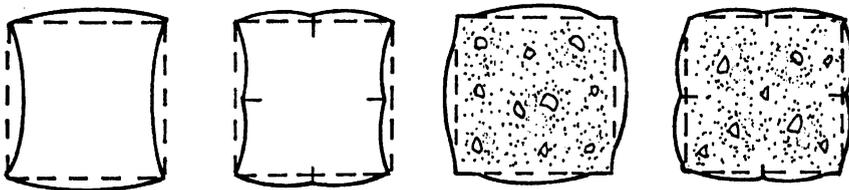

Fig. 2.   Buckling modes of steel and composite sections [14].

The stiffeners contributed largely to the overall buckling of columns even when stiffener rigidities were small, local buckling of longitudinal stiffeners being prevented by the concrete. In case of steel box columns, the local buckling of plate panels occurred before the maximum load. It was observed in composite columns that local plate buckling occurred initially in one of the plates of the columns just before the maximum load was reached, and the other buckling deformations took place in the remaining plates after peak. In addition, increase in deformations becomes faster after local buckling and cracks in weld occurred in some of concrete-filed columns.

5. Use of high strength concrete in composite columns

High strength steel has several advantages in its applications to tall buildings. Improvement in ductility of high strength steel has enhanced the research activities in this area. High strength and low weight are beneficial in seismic design with seismic response being reduced by the low weight of a structure. The stiffness in concrete increases with its characteristic strength. Higher strength concrete has an effective initial modulus of elasticity that increases roughly in proportion to the second or third root of the compressive strength and density [15]. Typical stress–strain curves of steel and concrete are given in Figs. 3 and 4, respectively. Rangan and Joyce [16] and O'Brien and Rangan [17] have reported the results of tests on eccentrically loaded slender steel tubular columns filled with high-strength concrete as high as 115 MPa. The eccentricity of the applied compressive load was equal at both ends, and the columns were subjected to single curvature bending. All specimens failed at mid-height due to crushing of concrete in the compression zone. In all specimens, the extreme fibre tensile strains at failure did not reach the yield strain of steel. The calculated ultimate loads of Rangan and Joyce [16] were found to
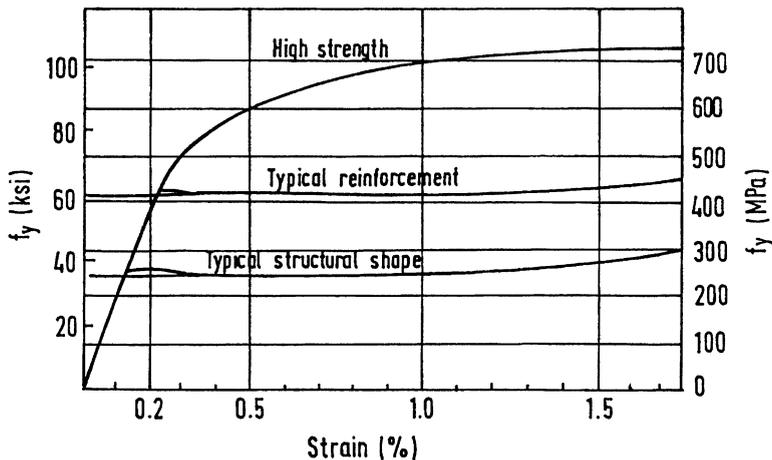

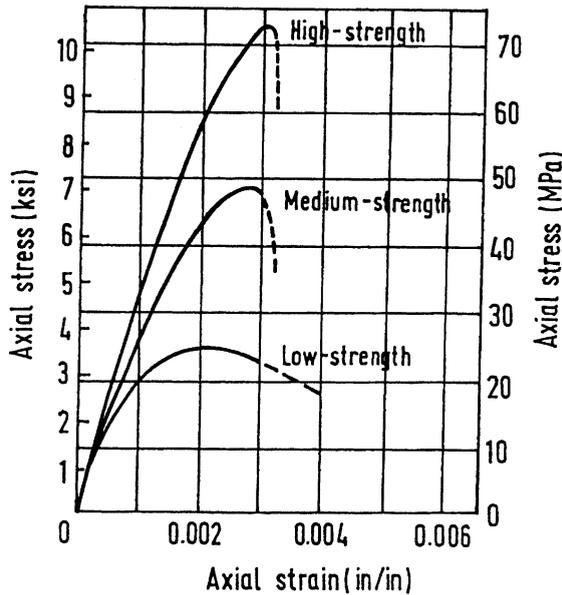Fig. 3.   Typical stress–strain curve for steel [15].

Fig. 4.   Typical stress–strain curve for concrete [15].

underestimate the experimental results with the maximum difference of 68%. Similar experiments were carried out to examine the behaviour of thin-walled circular steel tubes filled with ultra high strength concrete (115 MPa) [18]. The test specimens had an effective diameter to thickness ratio between 60 and 200 and a length to diameter ratio of 3.5. The specimens were tested under axial and eccentric loading. It was shown that unloading response of high strength concrete is rapid and may exhibit axial strain reversal, the snap-back process. Further test have been conducted to examine the potential enhancement in strength and possible improvement in ductility due to confinement of high strength concrete.

The high-performance concrete had less coarse aggregate and more powder materials than conventional air-entrained concrete. Although a higher ultimate strength was reached, it loses its stiffness sharply, unlike in columns filled with the ordinary concrete [14]. Deformation rate is high just after the peak, although the load is kept constant at a certain loading level, accordingly the stiffness decreased rapidly in concrete in-filled composite columns. Uy and Patil [3] presented a study on the behaviour of concrete filled high strength steel fabricated box columns. For a high strength steel box column filled with concrete, the steel usually may not have reached yield prior to the concrete reaching its ultimate strain which depends upon the yield strain of steel. Many advantages that can be attributed to the use of high strength steel in concrete filled steel box column construction were discussed. A cost comparison study shows that concrete filled box column construction can produce savings up to 50% on material costs alone when compared with conventional steel box columns and can also provide more useable floor area by reduction in column size.

6. Analytical studies

Ultimate load of a column can be defined as the highest load for which an equilibrium-deflected shape can be obtained, in other words the load at which the column stiffness becomes zero. Principal factors that make the determination of the ultimate load complicated are the non-linear material characteristics of both concrete and steel, geometrical imperfections and residual stresses in the steel section. It involves sub-division of the cross-section into a number of slices to allow the strain distribution to be idealized as a unique linear function. The forces in each slice may then be established and integrated to obtain the axial force. When the axial force converges to the applied load, the neutral axis is then defined. A typical strain distribution over the cross-section is characterized by the strain at the top fibre and the curvature. To analyse a section is to establish a load–moment interaction curve. Intersection of the loading line for an eccentrically loaded column with the strength envelope is then taken as the ultimate load. Ultimate strength of composite columns has been the subject of many investigations. A wide range of analytical methods are formulated to examine the applicability of composite columns under various loading conditions, summary of which is presented below.

6.1. Encased sections

Furlong [15] attempted to correlate an elliptical form of an interaction diagram between the axial load and moment and found the elliptical form too conservative. It was concluded that interaction function derived from the rules of the ordinary reinforced concrete ultimate shear strength approach is more reasonable. The flexural and axial stiffness were derived as an algebraic sum of stiffness of each component part as if they acted separately.

The finite difference method was employed by Munoz and Thomas Hsu [19] to establish the relationship between curvature and deflection. The effect of creep, twisting effects, axial and shear deformation and tensile stresses due to shrinkage was neglected. In addition, the initial imperfection and residual stresses were not included. Finite difference method in combination with the secant stiffness matrix was used to solve the system of nonlinear equations. Also design equation to predict the ultimate load capacity of short and slender concrete-encased square and rectangular composite columns under uniaxial and biaxial bending was proposed [20]. The design equation satisfied basic analytical and design parameters of both the American Concrete Institute (ACI) and the American Institute of Steel Construction (AISC). The second order effects on slender columns were considered by incorporating a moment magnification factor similar to the one used by the ACI for reinforced concrete columns with the appropriate adjustments for rectangular composite columns. The proposed uniaxial interaction equation is:

$$\left(\frac{P_\mathrm{n}-P_\mathrm{nb}}{P_0-P_\mathrm{nb}}\right)^\alpha + \left(\frac{M_\mathrm{n}}{M_\mathrm{nb}}\right) = 1 \tag{6}$$

where α varies from 1 to 3; M_n and P_n refer to a typical point in uniaxial interaction

diagram; M_{nb} and P_{nb} correspond to balanced condition and P_0 the maximum axial load.

An analytical method for computing the ultimate failure loads of encased composite columns, subjected to an axial load and biaxial end moments was presented by Virdi and Dowling [21]. As an integral part of the procedure, a rapid method of establishing the moment–curvature–thrust relations was described. The efficiency of the method stems from the use of gauss quadrature formulae. A second-order iteration technique, the generalized Newton-Raphson method, has been adopted to evaluate the deflection of the column at evenly spaced points along the column length. The ultimate load was obtained by establishing the highest load for which equilibrium at deflected shape can be obtained. Task Group 20 [22] of the Structural Stability Research Council (SSRC) proposed a specification for the design of steel–concrete composite columns in 1979, which was subsequently adopted in the 1986 AISC-LRFD code. This specification requires that steel-encased concrete sections be designed in a way that is similar to the design of steel columns, with modifications to the steel yield strength, modulus of elasticity, and radius of gyration to account for the effect of concrete and longitudinal bars. The specification also places limitations on the percentage area of steel, concrete strength and minimum thickness of the steel shell. The minimum thickness for steel shells in circular composite columns is calculated using the equation:

$$t \leq \sqrt{\frac{f_y}{8E_s}} \tag{7}$$

where f_y and E_s are the steel yield stress and modulus of elasticity, respectively.

Roik and Bergmann [23] proposed a design method for composite columns with unsymmetrical cross-sections. The method is applicable to cases of axial-compression and combined compression and bending. It was based on the simplified method for composite columns with symmetrical cross-sections given in Eurocode 4. However, the cross-sectional properties have to be calculated with respect to the elastic centroidal axis and the action effects with respect to the plastic centroidal axis.

A study into stiffening and strengthening effects obtained from a concrete encasement was reported by Stevens [24] who suggested a straight line interaction formula for the strength of beam-columns. The column curve generated by tests were compared with that of a reinforced column and it was concluded that due to the similarity in the behaviour of reinforced and encased columns, similar principles can be applied to the design concept.

Basu and Sommerville [25] carried out a theoretical study on large number of axially loaded composite columns and obtained a curve lying below the Perry-Robertson curve. Although the curve is satisfactory for practical purposes, it should be noted that, for large values of slenderness ratio, it implies that an encased section may have a lower capacity than the corresponding encased section. Roberts and Yam [26] suggest the use of an elliptic contour interaction surface for composite columns:

$$(M_x/M_{xu})^2 + (M_y/M_{yu})^2 < 1 \tag{8}$$

where M_x and M_y are the bending moments about x- and y-axes respectively. M_{xu} and M_{yu} refer to the ultimate moment of resistance about their corresponding axes.

6.2. In-filled sections

Strength calculation method proposed by Rangan and Joyce [16] assumed a sine function for the deflection of the column. In addition, the axial load capacity P_u of a slender eccentrically loaded steel tubular column filled with concrete was assumed to reach when the maximum moment M_u is equal to the ultimate bending moment M_n at the mid-height of the column. Deflection due to creep and imperfections in steel tube were treated as an additional eccentricity. Calculations of creep deflection are given as follows:

$$\text{Creep deflection } \Delta_{cp} = \Delta_{tot} - \Delta_e \tag{9}$$

$$\text{where } \Delta_{tot} = \frac{e}{\left[\left(\dfrac{P_c}{P_\phi}\right) - 1\right]} \tag{10}$$

$$\Delta_e = \frac{e}{\left[\left(\dfrac{P_{co}}{P_\phi}\right) - 1\right]} \tag{11}$$

in which

$$P_c = \pi^2 EI/L^2 \tag{12}$$

$$P_{co} = \pi^2 EI_{gt}/L^2 \tag{13}$$

P_ϕ is the axial thrust due to sustained loads; L is the effective length of a column,

$$EI = E_c I_{gt}/(1 + 0.8\phi_{cc}) \tag{14}$$

$E_c I_{gt}$ is the bending stiffness of the transformed uncracked concrete section of the column at mid height. ϕ_{cc} is the creep factor which is the ratio of creep strain to elastic strain in concrete.

Bradford [27] proposed the following load–moment interaction curve assuming the position of the ultimate neutral axis as a parameter.

$$\frac{\phi M_s}{\phi M_0} = 1 + 2\left(\frac{\phi N_s}{\phi N_0}\right) - 3\left(\frac{\phi N_s}{\phi N_0}\right)^\alpha \tag{15}$$

in which

$$\alpha = 1.0 + 0.5\lambda; \tag{16}$$

$$\lambda = \frac{0.85(b-2t)(d-2t)f'c}{2(b + d-2t)t\sigma_y}; \tag{17}$$

ϕM_s is the column central moment, ϕN_s the corresponding strength to ϕM_s, ϕM_0 the factored design strength in pure bending and ϕN_0 the factored axial strength.

ϕ the resistance factor and index α was obtained by matching the approximate curve for several different sections through assumed balance point. The method was found conservative compared to the test results.

A design formula with an empirical reduction factor that accounts for the effect of in-filled concrete prism size and the concrete strength class was proposed by Ge and Usami [28]. A hardening–softening model was used to describe rationally the elasto-plastic behaviour of concrete. Contact element for the interface combined with bilinear constrained shell elements for the plate and a cubic element for the concrete was employed. Both initial geometrical imperfections and residual stresses were also considered in the plate elements.

Local buckling strength of the plate panel in composite columns was then compared with available empirical design formulae for a thin-walled steel member in compression. Longitudinal stiffeners were identified to be effective against local buckling of plate panel. Stiffening effect tends to increase due to the bonding between the stiffener and concrete [14].

Wang and Moore [29] developed a simple design procedure suitable for manual calculation by replacing the properties of the bare steel section with those of the composite section, based on the recommendations given in BS 5950 for bare steel column. Though the proposed method was conservative, the simplicity was perhaps a major factor in its favour. In this method, there are two equations: one for local buckling capacity check:

$$\frac{N}{N_u} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1 \tag{18}$$

and the other for global buckling check:

$$\frac{mM_x}{M_{ax}} + \frac{mM_y}{M_{ay}} \leq 1 \tag{19}$$

M_{ax} and M_{ay} in the above equation are the reduced column maximum buckling moments about the major and minor axis respectively, in the presence of the axial load N. The values of these moment resistance can be related to the column plastic moment capacities M_{cx} and M_{cy} and the column compression resistance N_{cx} and N_{cy} in the following way:

$$M_{ax} = M_{cx} \frac{1 - \dfrac{N}{N_{cx}}}{1 + \dfrac{0.5N}{N_{cx}}} \tag{20}$$

$$M_{ay} = M_{cy} \frac{1 - \dfrac{N}{N_{cy}}}{1 + \dfrac{0.5N}{N_{cy}}} \tag{21}$$

In the above equations which follow the approach given in BS 5950, the values of

N_u, M_{cx}, M_{cy}, N_{cx} and N_{cy} correspond to composite column. Wang used column-buckling curves 'a' for concrete-filled columns, 'b' for concrete encased columns bending about the major axis and 'c' for concrete encased columns bending about the minor axis. Composite cross-section is transformed to an equivalent steel section. Area A_s and the second moment of inertia, 'I' of the equivalent steel section are given as:

$$A_s = \frac{N_u}{P_{ys}/\gamma_s} \tag{22}$$

$$I = I_s + I_c\frac{0.67E_c/r_c}{E_s/r_s} \tag{23}$$

in which the subscripts 's' and 'c' refer to steel and concrete components, respectively, and I_c is the second moment of inertia of the untracked concrete component. Factor 0.67 relates the uniform rectangular stress block to the concrete cube strength.

The elasto-plastic behaviour of pin-ended, concrete-filled tubular steel columns subjected to axial or uniaxial load was studied numerically by Neogi et al. [2]. The triaxial and biaxial effects of concrete were not considered. The concentrically loaded straight column was analysed by the tangent-modulus approach, whereas the eccentrically loaded column was analysed by determining the 'exact' deflected shape or by assuming the shape to be part of a cosine wave. Central deflection i.e., deflection at mid-height is related directly to the central curvature as:

$$\rho_0 = -\frac{4}{l^2}\left[\cos^{-1}\left(\frac{e}{e + \delta_0}\right)\right]^2 \cdot (e + \delta_0) \tag{24}$$

where δ_0 and ρ_0 stand for the central deflection and curvature respectively; e—eccentricity of load, and l—effective length of the column. Good agreement was observed between the experimental and theoretical behaviour of columns with l/d ratios greater than 15. It was concluded that the triaxial effect diminishes as the eccentricity increases.

A straight line interaction formula proposed by Knowles and Park [30] is an approximation which may be unsafe for slender columns and conservative for short columns. Experimental behaviour of hollow and concrete filled steel tubular columns appears similar without any reflection of concrete filling. Ben Kato [31] proposed the following design formulae based on ISO standards with appropriate modifications for both encased and in-filled column. Buckling strength of centrally loaded concrete-filled steel tubular columns is given by,

$$N_c = gN_y \tag{25}$$

$$N_y = A_s\sigma_y + \beta A_c f_c + (a_r\sigma_{yr}) \tag{26}$$

$\beta = 1.1$ for circular sections
$\beta = 1.0$ for square sections.

$$g = B(1-\sqrt{1-C}) \tag{27}$$

$$B = \frac{1 + 0.34(\bar{\lambda}-0.2) + \bar{\lambda}^2}{2\bar{\lambda}^2} \tag{28}$$

$$C = (B\bar{\lambda})^{-2} \tag{29}$$

$$\bar{\lambda} = \frac{kL}{\pi} \sqrt{\frac{A_s\sigma_y + \beta A_c f_c}{EI_s + E_c I_c}} \tag{30}$$

and for concrete-encased steel columns

$$N_y = A_s\sigma_y + 0.8A_c f_c + (a_r\sigma_{yr}) \tag{31}$$

$$\bar{\lambda} = \frac{kL}{\pi} \sqrt{\frac{A_s\sigma_y + 0.8A_c f_c + (a_r\sigma_{yr})}{EI_s + (EI_r) + E_c I_c}} \tag{32}$$

In the above equations, A_s—sectional area of steel section; A_c—area of concrete; f_c—compressive strength of concrete; kL—effective length of column; E_s—modulus of elasticity of steel; I_s—moment of inertia of steel section; σ_y—yield stress of steel; a_r—area of longitudinal reinforcing bars for concrete-encased steel column; σ_{yr}—yield stress of longitudinal reinforcing bars for concrete-encased steel column; I_r—moment of inertia of longitudinal reinforcing bars; I_c—moment of inertia of concrete; E_c—modulus of elasticity of concrete.

However, limited experimental and analytical methods on the response of high-strength steel tubes filled with high-strength concrete to short term loading are available. Effects of parameters such as slenderness, confinement, reinforcement and imperfections in the shape of the tube on the ultimate strength of composite section are to be investigated rigorously.

7. Shear resistance and bond strength

The use of mechanical connectors may be necessary in special circumstances in which the limiting bond stress is likely to be exceeded for example in the presence of significant transverse shear on the column, and also in the case of dynamic and seismic loading. For a smooth steel surface, the mechanical resistance is of less importance than for an embossed or irregular steel surface. The influence of an interface pressure on the force transfer is therefore more important for a smooth steel surface found in composite columns than for a surface with embossments or irregularities. Plain concrete without reinforcement will have no shear strength after either flexural or shear cracking, unless it is confined inside relatively short length of steel tube or pipe.

Suzuki and Kate [32] observed that in relatively short concrete-filled tubes or boxes, the confined concrete can act as a diagonal compression strut together with tension field action of the steel side walls to resist limiting shear that are significantly greater than the shear capacity of the steel sidewalls alone. Shakir-Khalil and Mouli [11] found from experiments bond strength to vary between 0.39 and 0.51 N/mm².

It is relatively low compared to those for reinforcenent bars and circular hollow sections. Moreover, because of the relative flexibility of its walls, the variation in the shape of the rectangular hollow section would have a less beneficial effect on the bond strength than in the case of a circular hollow section. Virdi and Dowling [21] showed that bond occurs through the interlocking of concrete by two types of imperfections, namely surface roughness of steel and the variation in the shape of the tube cross-section. In the case of concrete-encased composite columns it is recommended that a reinforcement cage should be used to contain the lateral expansion of concrete. This also prevents the premature spalling of the concrete encasement, especially the thin concrete cover to the flange of an encased I-section.

8. Ductility

It is often necessary to determine the ductility or rotational capacity of a given column. The curvature ductility, defined as the ratio of curvature at ultimate load to the curvature at yield, can be obtained analytically by studying the moment curvature relationship. Kitada [33] described the difference in local buckling modes between cross-sections of steel and composite columns, the difference in cross-sections between composite columns in bridge piers and buildings. It is observed that the ductility of the composite beam-column specimen with rectangular cross-section is small compared to that with a circular cross-section in the case of large axial compression (Fig. 5).

To determine the curvature ductility of composite steel concrete columns and to investigate on the adequacy of their use in seismic areas, a study was conducted by Itani [34]. Parameters such as thickness of steel jacket, D/t ratio and the percentage of longitudinal reinforcement were considered in the analysis. To ensure full composite action between the steel jacket and the concrete, shear studs were provided throughout the length of the column.

9. Seismic behaviour

Partially-encased composite steel–concrete beam-columns under cyclic and pseudo-dynamic loading was presented by Elnashai and Elghazouli [35,36]. Provision of additional transverse bars intended to inhibit local buckling at large displacements and to increase the interaction between the two materials was considered in the study. The modified section seemed to be showing significant improvement in the ductility and energy absorption capacity under cyclic and transient dynamic loading. After the shock of Hyogo-Ken Nambu earthquake, a method of inserting an additional steel tube into the steel bridge pier was considered by Nakanishi et al. [37]. In this method, the ductility of the bridge pier was significantly enhanced if the cross-section was designed such that the axial compressive load caused by dead load of the superstructure was mainly carried by the inner tube. It was found from the study that the natural period of vibration for bridge piers affect significantly the
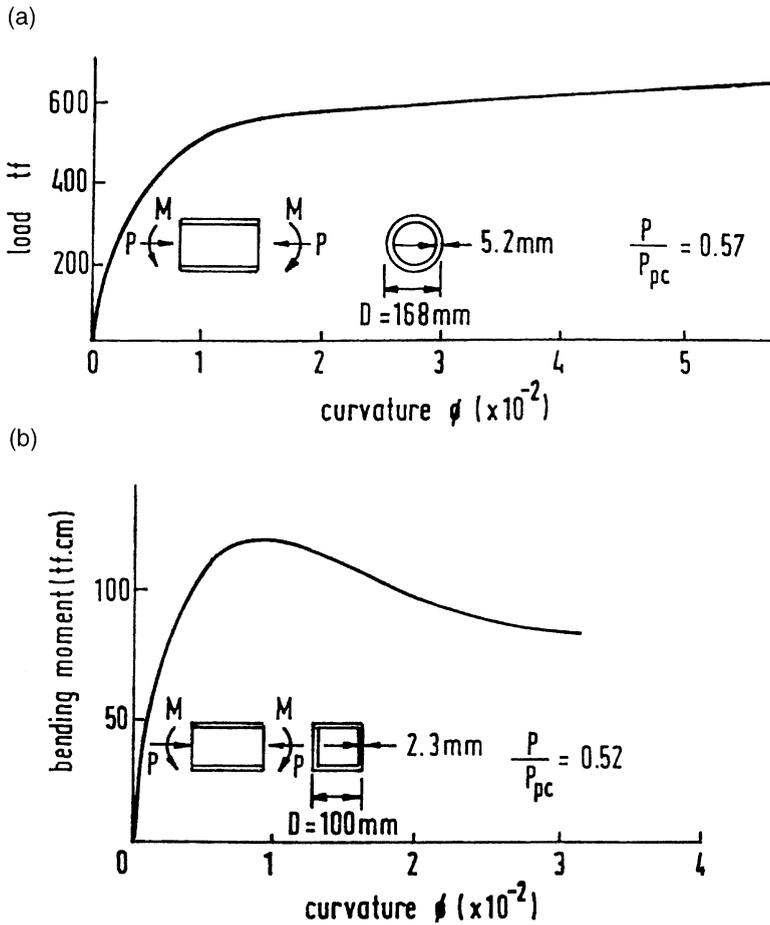
(a)

(b)

Fig. 5.   Moment–curvature plots for short composite specimens [33]. (a) Circular cross-section; (b) square cross-section.

response of displacement under earthquakes and the residual displacement after earthquakes.

The deterioration of hysteresis curves is more severe with the increase of slenderness. Local buckling occurred at early loading cycles whereas fracture required many cycles after local buckling. Boyd et al. [38] reported based on results from the investigation of flexural behaviour comparing the performance by columns of same D/t ratio, columns with different D/t ratios, columns with studded and non-studded shells and columns with normal and high-strength concrete cores. The ductility for columns with studded and non-studded steel shells was found to be similar. Local buckling in the steel shell and concrete cracking in the core cause irregularities in the load–displacement hysteresis curves. A definition of energy dissipation is outlined in Fig.

6. It was observed that the energy–dissipation ratio was higher than that for the conventionally reinforced columns [38].

The behaviour of short and intermediate length square plain concrete-filled steel columns subjected to cyclic lateral forces with a constant axial load was examined. The resulting hysteretic loops indicated a stable response with a considerable amount of energy dissipation and some strength degradation. The strength degradation was a result of local buckling in the steel shell, leading to crushing of the encased concrete [39,40].

Park et al. [41] conducted tests on a number of steel-encased plain and reinforced concrete sections with D/t ratios of 34–214 subjected to cyclic lateral load and constant axial load. The performance of the columns was shown to be affected by local buckling that developed in the casing at the critical sections, and fracture occurred after several succeeding cycles of lateral load. When the variable repeated loading (VRL) exceeds certain limit, it causes excessive inelastic deformation that grows with repetition of the load and eventually leads to incremental collapse. An experimental investigation was carried out involving parameters such as concrete strength, end moment, and effect of filling [42]. The incremental collapse limit lies between 70% and 79% of the static collapse load for composite columns. An improved system of detailing which offers advantages in terms of efficiency, economy of fabrication and enhanced seismic performance was suggested.

10. Effect of local buckling

Thin-walled circular composite columns used in many constructions have to be designed to account for the confinement effect of concrete restraint against local buckling of steel tube. Design of the steel casing using a rational analysis for local
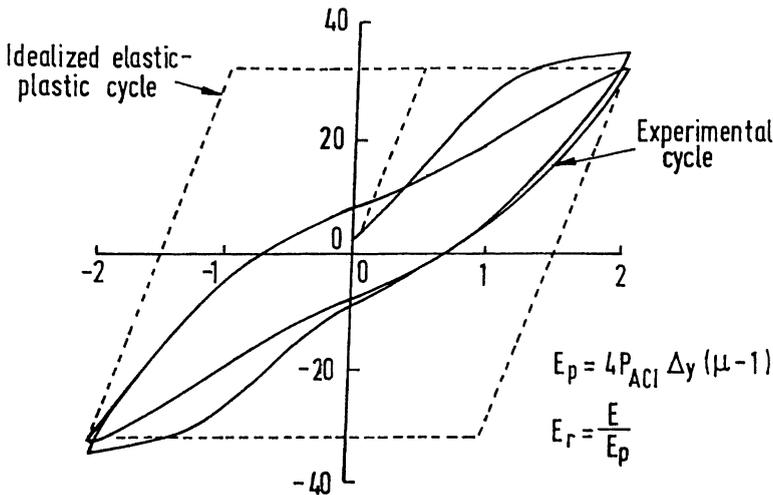

Fig. 6.   Definitions of energy dissipation [38].

buckling would lead to considerable saving on material cost. A concrete-filled tube has a local buckling capacity of about 50% more than that for unfilled tube since the steel tube is restrained against buckling inwards by the concrete infill [43]. Effect of local buckling on the axial compressive strength of circular steel tubes is a function of the plate diameter to thickness ratio (D/t) and is accounted in a number of design standards through the use of an effective diameter or an effective area.

A model for local buckling of steel plates when in contact with a rigid medium was developed by Wright [44]. His theoretical model is based on the energy method but is applicable to uniform compression only. A semi-analytical procedure to incorporate the elastic and in-elastic local buckling of plates with clamped loaded edges were proposed using cubic polynomial [45,46]. A post local buckling model based on the effective width principle was established [47]. The local buckling stress was set equal to the yield stress for columns that buckled inelastically. Local buckling strain was determined as the point at which a significant change occurs in the average load–strain relationship. The model seems to be very accurate in the elastic region with residual stresses in the order of 30%. However, the finite strip analysis does not incorporate initial imperfections which play a significant role in reducing the local-buckling load in thin-walled structures.

11. Long-term effects

The physical properties of concrete are such that it contracts while setting followed by a lengthy period of shrinkage and of creep under load. Increasing lateral strain accompanied by transfer of stress from concrete to steel results in reduction of ultimate load capacity of slender columns subjected to eccentric load. In theory, shrinkage of the concrete is more effectively restrained in steel–concrete columns than in ordinary reinforced concrete columns. Because of the humid conditions inside the steel section, the coefficient of contraction is relatively low and shrinkage occurs very slowly.

A design procedure was proposed by Bradford and Gilbert [48] for estimating the maximum acceptable service load on a slender, eccentrically loaded composite column. It is found that the loss of strength due to long-term effects such as creep and shrinkage is insignificant; however their effects are yet to be quantified. Due to creep under sustained stress, concrete does not have a constant value for its modulus of elasticity. The stiffness EI of cross-sections under initial applications of service loads (relatively low stress values) can be taken as the sum of flexural stiffnesses for all component parts of the section:

$$EI = E_s I_s + E_c I_c \tag{33}$$

The value of I_c will diminish after the concrete cracks in flexural tension, and the effective value of E_c will be reduced from long-time loading at high levels of compression stress. Under sustained loading, creep of concrete results in increase in the lateral deflection of a composite column and a reduction in its strength and stiffness. Basu and Sommerville [25] gave magnification factors to be applied to that part of

the loading which was considered to be sustained. These factors were functions of column slenderness, area of concrete and its disposition in the cross-section. They also allowed for the effect of shrinkage and creep by increasing the instantaneous strain level in the concrete by a factor of two.

12. Effect of confinement

Circular hollow sections provide a significant amount of confinement while this effect is negligible in the case of rectangular sections. Additional strength occurs because of the increase in compressive strength of the concrete core that is restrained laterally by the surrounding steel tube. This increase in concrete strength outweighs the reduction in the yield strength of steel in vertical compression due to the confinement tension needed to contain the concrete. The confinement effect is not present in concrete-filled rectangular hollow sections, except in the corner regions, as the hoop tension developed along the side walls is not constant [49].

In the concrete-filled circular sections, the influence of containment is reduced as bending moments are applied. This is due to the mean compressive strain in the concrete (and the associated lateral expansion) is then reduced. It also diminishes with increasing slenderness of the column, since the lateral deflection prior to failure increases the bending moment and reduces the mean compressive strain in the concrete. The confinement may occur for columns where concrete is crushed prior to local buckling of steel and this would generally be true for columns where the plate slenderness limit is small. The effect of confinement was calculated in terms of confining pressure as circumferential stress f'_1 [50]. Steel jacket would be subjected to biaxial state of stress as shown in Fig. 7.

Longitudinal stress, 'f_s' developed due to axial load and bending moment, whereas a circumferential stress, 'f_1' develops due to concrete confinement. The two stresses define the yield criteria as outlined by von Mises' yield criteria.

$$f_s^2 + f_1'^2 + f'_1 f_s = F_y \tag{34}$$

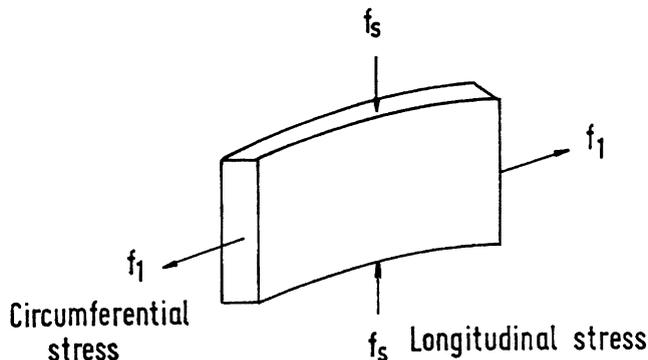where 'F_y' is the yield stress of steel jacket.

Fig. 7.   Longitudinal and circumferential stresses in the steel tube [50].
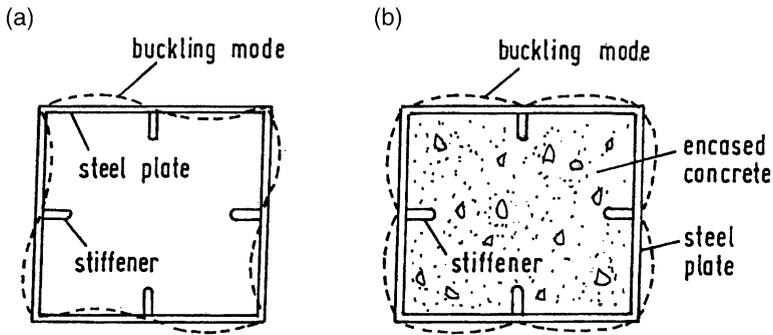
Fig. 8.   Buckling modes of steel and composite sections [33]. (a) Steel cross-section; (b) composite cross-section.

13. Comparison with steel columns

The basic buckling modes of steel and composite columns are illustrated in Fig. 8 [33]. In the case of concrete-filled steel tubular columns, concrete inside the tube prevents inward-buckling modes of the steel tube wall, and the tube-wall in turn provides effective lateral confinement to the concrete inside the tube. Typical examples of load–average strain curves for steel and concrete-filled column are shown in Fig. 9.

The unloading response of the tubes becomes rapid in case of composite columns, with increasing tube wall slenderness due to local buckling. The response of composite tube is similar to that for the bare steel tubes. The hysteretic loops in the concrete filled columns were relatively narrow in the early cycles, and then become wider at the later cycles due to strain hardening in the post-ultimate region. The maximum strength is obtained at about a strain of 0.2% in the steel box columns and in the range of 0.3–0.4% in the concrete-filled columns. Obviously, the
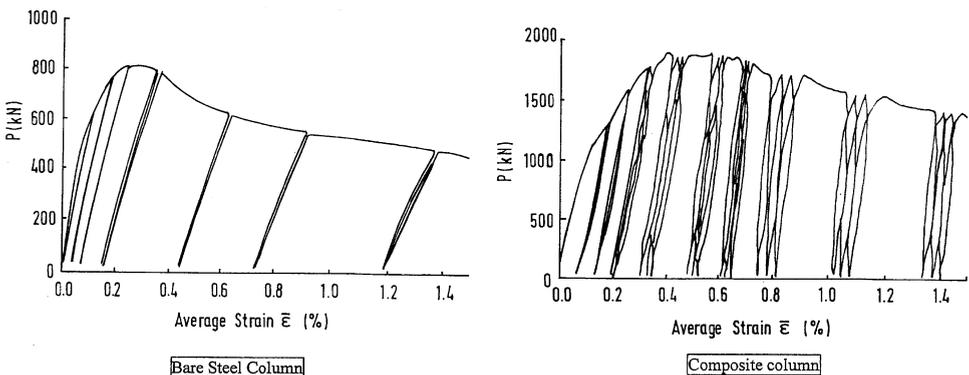

Fig. 9.   Load–average strain curves under cyclic loading [14].

maximum strength of concrete-filled columns was much larger than those of the steel columns. Therefore, it can be concluded that the concrete-filled column shows good structural performance through higher ductility and higher strength [14].

14. Design codes

Over the last two decades, researchers have suggested analytical methods and design procedures for composite columns and design codes have been formulated. Each of these codes is written so as to reflect the design philosophies and practices in the respective countries. Over the last two decades, different specific codes for the design of concrete filled steel tubular columns have been used.

14.1. The building code requirements of reinforced concrete (ACI 318-89)

According to ACI 318-89, a composite column is a concrete column reinforced with a structural steel shape or tubing in addition to reinforcing bars. In order to consider the slenderness effects, an equivalent radius of gyration and flexural stiffness are used with a parameter of sustained load ratio, and hence without any sustained load, radius of gyration should be taken as zero. The limiting thickness of steel tube to prevent local buckling are based on achieving yield stress in a hollow steel tube under monotonic axial loading which is not a necessary requirement for in-filled composite column. A parameter for the softening influence of creep in concrete that is subjected to sustained compressive loading is included.

14.2. Load and resistance factor design method (AISC-LRFD)

This is based on the same principles as ACI code. Design is based on equations for steel columns. Nominal strength is estimated on the basis of ultimate resistance to the load, and reduction factors are then applied. The nominal axial load capacity is reduced according to the slenderness ratio. Neither the ACI-318 nor the AISC-LRFD provisions explicitly consider confinement effects on strength or ductility of member analysed. ACI provisions for calculating the strength interaction between axial and flexural effects are essentially the same as those for reinforced concrete column, whereas AISC-LRFD are based on the bilinear interaction formulae which have the same form as those of steel columns. In the above design methods, flexural stiffness is underestimated and confining effect of the steel tube on the concrete core is ignored. The influence of creep is ignored for concrete in composite columns according to AISC-LRFD specification.

14.3. Architectural Institute of Japan (AIJ)

A composite structural system using concrete and steel shape is called 'steel reinforced concrete' (SRC) in Japan. The allowable stress design is primarily employed, in which working stresses are calculated based on the elastic stiffness of

members and allowable strength by the superposed strength formulae. Cross section strength is calculated by superimposing the strength of both the steel and concrete sections, thereby neglecting the interaction between steel and concrete and the effect of confinement. Euler buckling load is used with a reduced concrete stiffness and factors of safety for both concrete and steel. The method is applicable to asymmetrical sections and columns under biaxial bending.

14.4. British Standard BS 5400—Part 5

Code provisions in BS 5400 are based on limit state design with loading factors and material safety factors. The ultimate moment is calculated from plastic stress distribution over the cross-section, and an approximation for the interaction curve for axial load and moment is used. Reduced concrete properties are used to account for the effects of creep and the use of uncracked concrete section in stiffness calculation. This method is applicable to symmetrical sections only and restricted to the range of sections catered for in the European buckling curves. It underestimates the capacity of in-filled composite columns with high-strength concrete.

14.5. European Code EC4

EC4 covers concrete encased and partially encased steel sections and concrete filled sections with or without reinforcement. This code uses limit state concepts to achieve the aims of serviceability and safety by applying partial safety factor to loads and material properties. Based on experimental results, it was recommended that the regulations of EC4 concerning the factor of 0.85 should not be applied to hollow sections filled with high strength concrete [23]. This is the only code that treats the effect of long-term loading separately.

All codes assume full interaction, but some impose restrictions on the shear stress at the steel–concrete interface. It is customary to use direct bearing or provide shear connectors, if used where the specified limiting shear stress is exceeded.

15. Conclusions

Considerable progress has been made during the last two decades in the investigation of steel–concrete composite columns, and information available is summarized in this paper. The details of experimental works available are given in Table 1 for encased composite columns and in Table 2 for in-filled composite columns. The tables provide information such as number of tests, section shape, loading type, variables considered in the study and the origin of work, etc. Fundamental knowledge on composite construction system such as ultimate strength, has already been obtained by the research carried out so far. Intensive research is required on the interaction between steel and concrete, the effect of concrete restraining local buckling of steel plate elements, effect of steel section, confining concrete, etc.

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