Buckling of Concrete-Filled Steel Tubular Slender Columns

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ABSTRACT

Buckling of composite columns under axial loading is the main objective of this research. So for this reason a lot of tests on steel tubular columns of rectangular and circular section filled with normal and lightweight concrete were conducted to investigate the buckling behavior of such columns under axial loadings. Comparisons between Normal and lightweight concrete filled steel columns for different columns cross sections using Euro Code 4 and BS 5400 codes were carried out. The test results showed that both types of concrete filled columns failed due to overall buckling, while hollow steel columns failed due to local buckling at the ends. According to the above-mentioned results the further interest should be taken onto the replacement of the normal concrete by any type of lightweight concrete due to its low specific gravity and reasonable bearing.

Keywords: *Buckling of Columns, Steel Columns, Tubular Columns, Lightweight Concrete, Normal Concrete, Local Buckling, Overall Buckling.*

1. INTRODUCTION

In slender columns, where buckling will occur, the steel shell will add significantly to the strength. When the concrete-filled steel tubular columns are employed under favorable conditions, the steel casing confines the core and the filled concrete inhibits local buckling of the shell. However, thermal conductivity of lightweight concrete, as well as the low specific gravity that produces lighter structures, seems to be good reasons for using lightweight concrete in composite construction. Several investigations carried out by Khalil and Brauns (1998) conducted a stress analysis from concrete-filled steel tubular columns. His recommendation was summarized in the following conclusion: In order to prevent the possibility of column failure in the case of small steel thickness, large eccentricities and suitable steel strengths have to be used.

Tests were conducted by Wang (1999) on concrete filled rectangular hollow steel slender columns . They were loaded with end eccentricities producing moments other than single curvature bending. Y.M.Hunaiti (1997), conducted an experimental study on steel hollow tubes of square and circular section filled with foamed and lightweight aggregate concrete, and he concluded that the foamed concrete-filled column specimens were incapable of reaching the predicted values of the squash load, while column specimens filled with lightweight aggregate concrete developed the ultimate axial capacity and the lightweight concrete enhances the strength of the steel section. Brauns (1998) conducted a stress analysis for concrete-filled steel tubular column.

The purpose of the present study was to study the buckling behavior of different types of filled concrete columns and to do a comparison between the tests and the existing design codes using ***Euro Code 4 and BS 5400 codes*** .

2. EXPERIMENTS

Twelve full scale column specimens of rectangular, and circular steel hollow sections, designated R for rectangular, and C for circular, were tested in this study. All columns were slender with various lengths and slenderness ratios and of cross-sectional dimensions as shown in Fig.1, and Table 1.

The column specimens comprised three different groups. first group specimens consisting of four specimens were filled with lightweight aggregate concrete (designated LWC), and the second group specimens also consisting of four specimens, were filled with normal weight concrete (designated NC). The rest of the column specimens were tested as bare sections for comparisons (HS). Designation and sectional properties of the specimens are given in Table 2.

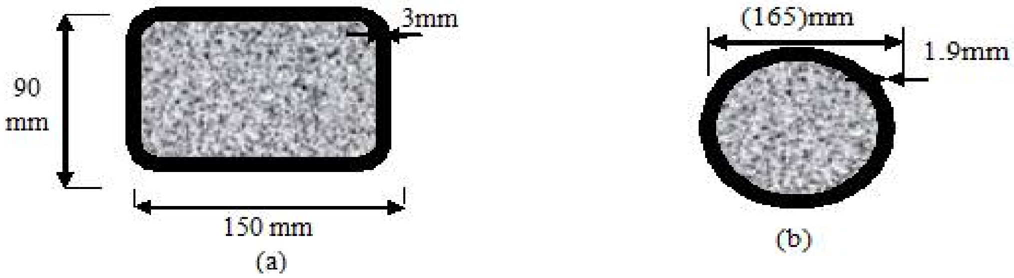

Fig. 1 Cross-Sectional Dimensions of Test Specimens: (a) Concrete-Filled RHS; (b) Concrete-Filled CHS

The columns were of different sizes, shapes, lengths and slenderness ratios. From the prototype sections of 200x100x5mm, 150x90x3mm, 110x1.9mm and 165x4.7mm. Three specimens of each sections were prepared , one of them was filled with normal concrete and another was filled with lightweight concrete, but the last one was tested as a hollow steel section. End plates, 8mm thick. Were welded to the column ends by 5mm fillet welds.

Table 1 : Designation and Sectional Dimensions of Some Specimens

Column Designation	Section Dimensions (mm)	Effective Length (mm)	Depth	Width	Thick	Dia.	Slend Ratio
C-N.C	200x100x5 Rectangular	2100	200	100	5	…	15
C-LWC	200x100x5 Rectangular	2100	200	100	5	…	15
C-H.S.	200x100x5 Rectangular	2100	200	100	5	…	15
C-N.C.	150x90x3 Rectangular	2500	150	90	3	…	25
C-LWC	150x90x3 Rectangular	2500	150	90	3	…	25
C-H.S.	150x90x3 Rectangular	2500	150	90	3	…	25
C-N.C	165x4.7 Circle	2475	…	…	4.7	165	15
C-LWC	165x4.7 Circle	2475	…	…	4.7	165	15
C-H.S.	165x4.7 Circle	2475	…	…	4.7	165	15

C-N.C.	**110 x 1.9 Circle**	**2200**	**…**	**…**	**1.9**	**110**	**20**
C-LWC	**110 x 1.9 Circle**	**2200**	**…**	**…**	**1.9**	**110**	**20**
C-H.S.	**110 x 1.9 Circle**	**2200**	**…**	**…**	**1.9**	**110**	**20**

Two different concrete mixes were used with a maximum size of aggregate of 10mm. For normal concrete, a concrete mix of 1 : 1.4 : 2.8 / 0.6 was used. Ordinary Portland cement, medium crushed limestone aggregate gravel and fine sand (2mm size) were used. For the lightweight aggregate concrete, pumice of 10mm size was used with expanded perlite. Proportions suggested by (Sabaleish, 1988) were used to produce the lightweight concrete. Details of the concrete mixes and material properties of the columns are summarized in Table 2 and Table 3.

Table 2 : Details of the Concrete Mixes

Type of Concrete	Cube Strength, f_{cu} (Average Value *MPa*)	Density, ρ (Average Value (Kg/m^3)	Concrete Mix Proportions
Normal Weight Aggregate Concrete	33.4	2081	Cement : Sand : Agg. 1 : 1.4 : 2.8 *w/c* = 0.6
Lightweight Aggregate Concrete	10	1390	Cement : Pumice 1 : 1.53 Expanded Perlite: 0.92 *L/kg* of Pumice *w/c* = 0.85

Table 3 : Details and Section Properties of Columns

Steel Section	Dimensions of Section (mm)	Area of steel (mm2)	Area of concrete (mm2)	Yield Strength (Mpa)	Mod. of Elasticity (Mpa)
Rectangular	200x100x5 150x90x3	2900 1404	17100 12096	360 320	229300 201000
Circular	165x4.7 110x1.9	2267 645	19016 8858	355 350	227000 220100

The column specimens were tested under incremental monotonic loading in a 2,000-kN capacity compression hydraulic jack (M1000/RD), with a deformation rate of 0.01mm/sec. All specimens were prepared and placed under the applied load with a high degree of accuracy to ensure the load application to the required positions.

3. DESIGN CONSIDERATIONS

The ultimate load-carrying capacity of a composite column can be calculated by several methods, which exist in codes of practice. The Bridge Code (BS 5400,1979), and the Eurocode4, 1985 contain rules of the design of composite columns. These rules are applicable only to concrete-filled steel tubes and to concrete-encased steel sections.

In calculating the squash load [defined as the ultimate short term axial load for short column], Nu, according to:

The Bridge Code and Eurocode 4 for

A. Rectangular .,or Square Sections are given as:

$$N_u = A_s f_{sk} / \gamma_{ms} + A_c f_{ck} / \gamma_{mc} \qquad (1)$$

The material partial safety factors for steel and concrete γ_{ms} and γ_{mc} were taken as unity. Moreover, the value of the characteristic concrete strength f_{ck} was ***taken as :***

$$f_{ck} = 0.83 f_{cu} \qquad (2a)$$

$$\text{instead of} \quad f_{ck} = 0.67 f_{cu} \qquad (2b)$$

Where fcu is the 28 day cube strength of concrete.

The value of 0.83f cu is recommended by *EC4* for experimental work. Furthermore, the ratio between $A_c f_{ck} / \gamma_{mc}$ and N_u is called the concrete contribution factor, α , and for a filled composite section it should vary between 0.1 and 0.8 . Also the characteristic steel strength f_{sk} was taken as: $f_{sk} = 0.91 f_y$.

B. Circular Sections: The squash load is given as:

$$N_u = 0.91 A_s f_y` + 0.45 A_c f_{cc} \qquad (3)$$

In which, the enhanced concrete characteristic strength***:***

$$f_{cc} = c_1 f_{yt} / D_e + f_{cu} ,$$

and the reduced yield steel strength***:***

$$f_y` = c_2 f_y$$

Where: c1 and c2 constants depend on column length and its diameter . Also the concrete contribution factor,

$$\alpha_c = 0.45 A_c f_{cc} / N_u$$

but according to Eurocode 4, the plastic resistance load,

$$N_{plrd} = A_a f_y / \gamma_a + A_c f_{ck} / \gamma_c$$

In an axial loaded slender column where length to least dimension of the cross section (L / b) should be greater than 12, failure occurs due to buckling about the minor axis and initial imperfections in straightness of the steel member. In practice, end moments due solely to the load acting at, an eccentricity may arise from construction tolerances.

The design methods for axially loaded columns therefore include an allowance for an eccentricity about the minor axis not exceeding 0.03 times the least lateral dimension of the composite column (b). The design load acting on the column, N_d , is not greater than uniaxial load [Min moment included in the design for slender columns due to imperfections] N_y , which is given by :

$$N_y = N_u [k_{1y} - \{k_{1y} - k_{2y} - 4k_3\}\{ M_y / M_{uy} \} - 4 k_3 \{ M_y / M_{uy} \}^2] \qquad (4)$$

Where, k : constant with appropriate subscripts.

However , according to Eurocode 4, the design load , Nsd ,or the experimental load, *Nexp* , should be less or equal to χN_{plrd} , , in which, χ , is a reduction factor due to slenderness of the column.

Based on the rectangular full plastic stress distribution . Hunaiti, 1997, the ultimate moment of resistance of a concrete filled rectangular hollow section can be calculated from the following equation:

$$M_{uy} = f_{sk} [0.5 A_s (h` - d_{cy}) + bt (t + d_{cy})] \qquad (5)$$

where; A_s : area of steel cross section.

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h\` : depth of concrete cross section.
b : breadth of column cross section
t : thickness of steel column.

dcy , is the depth of the neutral axis and given by:

$$dcy = (Ast - 2bt) / (\rho h` + 4t) \qquad (6)$$

and, ρ, is the ratio of the stresses, and is given by:

$$\rho = fck / fsk \qquad (7)$$

Based on the rectangular full plastic stress distribution , the ultimate moment of resistance of concrete filled Circular HS sections (in minor axis) can be calculated from the following equation:

$$Muy = fsk . S(1 + 0.01m) \qquad (8)$$

Where, S , is the plastic section modulus of the composite column.

m is given by :

$$m = \frac{100}{S}\left[t(De - t)2(\beta \sin\beta + \cos\beta - 1) + \frac{1}{4} \rho \frac{(De - St)3\omega}{} \right] \qquad (9)$$

Where ,

$$\omega = \frac{1}{3}\cos 3\beta - \frac{1}{4}\sin\beta(\pi - \sin 2\beta - 2\beta) \qquad (10)$$

The depth of the neutral axis, or cosine the angle β , can be determined from the equilibrium conditions of the compressive and tensile forces, as defined by the stress distribution . Also m: can be determined by (BS 5400 : Part 5) which depends on depth to thickness ratio (De / t) and ρ [is the ratio of stresses which was defined before].

4. NUMERICAL RESULTS AND DISCUSSION

The column specimens behaved very well under load, and as shown in the Table 4 , the experimental failure loads of all column specimens were mostly well in excess of design values estimated by most composite codes. Eurocode 4 , as well, underestimates the failure loads of the bare steel sections. Design values together with experimental results are shown in Table 4. The results of the tested columns are presented as in the following procedures:

a. Sections filled with lightweight aggregate concrete failed due to local as well as overall buckling, and they were capable of supporting more than 92% of the squash load. The ratio between experimental and design values ranges from 104% to 130%.

b. Sections filled with normal concrete failed due to overall buckling at sidelight , and they were capable of supporting more than 87% of the squash load. Design code values of failure loads, according to all design codes, are also compared with the experimental results. The ratios between the experimental failure loads to the design loads vary between almost 100% and 138%.

c. Bare steel sections failed due to excessive yielding and bulging (local buckling) at both top and bottom ends of the column specimens before reaching the plastic load, and they were capable of supporting more than 88% of the plastic load. The ratios between the experimental failure loads and the design loads range from 95% to 122%.

All columns were tested under axial load. It can be seen from the load-deflection curves that the horizontal deflections in the major axis direction were very small and started to increase at loads more than 80% of the failure load.

Although both Eurocde 4 and the Bridge code take into consideration the enhancement of the strength of circular columns due to confinement, the Bridge Code predictions of the column strength (design code values) appear to be lower than that of Eurocode 4. It can obviously be seen that normal concrete-filled tubular columns support higher loads than those filled with lightweight aggregate concrete. Moreover, in terms of the cube strength, columns of more than three times stronger normal concrete compared to the lightweight concrete (cube strength of normal concrete is 33MPa, while it is 10MPa for lightweight concrete, about 3.3 times greater, while concrete contribution factor ratio, α , is 2.89) showed enhancement of the loads of only about 24%, but the weight of the column with lightweight concrete is lighter than that with normal concrete of the same cross section by about 26%. This leads to reduce the column sections.

5. CONCLUSIONS

The steel tubes filled with lightweight aggregate concrete and normal concrete show acceptable strength under the applied load when compared to design calculations. According to the experimental and design code calculations, the buckling behavior of both lightweight concrete-filled steel tubular column and normal concrete-filled steel tubular column is very similar.

Columns filled with lightweight aggregate concrete exhibited local buckling, and when the column reached failure load an overall buckling took place as shown in fig 2. Nevertheless, such negative effect (the local buckling) did not significantly reduce the load carrying capacity of the column . However columns with normal concrete exhibited overall buckling with no signs of local buckling prior to failure. It can be seen from the results of comparisons between different types of columns and different dimensions. Moreover, sections with larger dimensions exhibited higher load carrying-capacity. According to the above-mentioned results there is a good possibility of normal aggregate concrete replacement by lightweight aggregate concrete due to its low specific gravity.

Table 4 : Designation and Results of Some Specimens

Col. Desig-nation	C.Cont Factor α [BS]	C.Cont Factor α [EC4]	Squash Load (kN) [BS]	Squash Load (kN) [EC4]	Exp. Load (kN)	Design Load (kN) [BS]	Desig. Load (kN) [EC4]
C-N.C 200X100X5	0.303	0.303	1356	1356	1242	1089	1190
C-LWC 200X100X5	0.139	0.139	1103	1075	1062	885	991
C-HS 200X100X5	----	---	1050	1050	932	860	964
C-N.C 165X4.7	0.541	0.406	1498	1287	1058	1143	1149
C-LWC 165X4.7	0.376	0.184	1151	895	834	887	862
C-HS 165X4.7	---	---	836	836	763	670	771

Figure 2 : Mode of Failure for Some of the Tested Columns

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