Experimental Study on Light Weight Concrete-Filled Steel Tubes

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ABSTRACT

Tests on steel tubular columns of rectangular and circular sections filled with normal and lightweight concrete were performed to investigate the behavior of such columns under axial loadings. Comparison between normal and lightweight concrete filled steel columns for different column cross-sections using Euro Code 4 and BS 5400 codes was also conducted. The test results showed that both types of filled columns failed due to overall buckling; while hollow steel columns failed due to local buckling at the ends. According to these results, further interest was taken onto the replacement of normal concrete by lightweight concrete due to its low specific gravity and thermal conductivity.

KEYWORDS: Composite columns, Steel columns, Tubular columns, Lightweight concrete, Normal concrete, Local buckling, Overall buckling.

INTRODUCTION

It is well known that the performance of laterally confined concrete with respect to its strength and ductility is better than that of unconfined concrete. Composite columns form a very important application of composite constructions. The use of composite columns results in reduction in column size providing substantial benefits where floor space is at a premium such as in car parks and office blocks. Concrete-filled steel tubular columns have an advantage over spirally reinforced concrete columns. In the latter, the core and the cover behave like two significant savings in column size which could lead to significant economic savings. The different layers and the spiral do not come into action until the cover spalls off; whereas, in the former the core and the tube form one continuous homogeneous medium.

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Also, 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, the thermal conductivity of lightweight concrete as well as the low specific gravity that produces lighter structures seem to be logic reasons for using lightweight concrete in composite construction.

Several studies were carried out by Brauns (1998) to investigate a stress analysis of concrete-filled steel tubular columns. His recommendation was summarized as: "In order to prevent the possibility of column failure (in case of small steel thickness), large eccentricities and suitable steel strengths have to be used".

Wang (1999) conducted several tests on concrete filled rectangular hollow steel slender columns. They were loaded with end eccentricities producing moments other than single curvature bending. Hunaiti (1997) performed an experimental study on steel hollow tubes of square and circular sections filled with foamed and lightweight aggregate concrete. 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 lightweight concrete enhanced the strength of the steel section.

Table 1. Designation and Sectional Dimensions for Some Specimens

Column Designation	Section Dimensions (mm)	Effective Length (mm)	Depth (mm)	Width (mm)	Thickness (mm)	Diameter (mm)	Slenderness 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-N.C	165x4.7 Circle	2475	…	…	4.7	165	15
C-H.S	150x90x3 Rectangular	2500	150	90	3	…	25
C-LWC	165x4.7 Circle	2475	…	…	4.7	165	15
C-H.S	165x4.7 Circle	2475	…	…	4.7	165	15
C-N.C	110x1.9 Circle	2200	…	…	1.9	110	20
C-LWC	110x1.9 Circle	2200	…	…	1.9	110	20
C-H.S	110x1.9 Circle	2200	…	…	1.9	110	20

The purpose of the present study is to present a comparison between the tests and the existing design codes using Euro Code 4 and BS 5400 codes.

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. The first group of specimens consisted of four specimens that were filled with lightweight aggregate concrete (designated LWC). The second group of specimens also consisted of four specimens. They were filled with normal weight concrete (designated NC). The rest of the column specimens were tested as bare sections for comparison (HS). Designation and sectional properties of the specimens are given in Table 2.

Table 2. Details for the Concrete Mixes

Concrete Type	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: Aggregate 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 for Columns

Steel Section	Dimensions of Section (mm)	Area of Steel (mm^2)	Area of Concrete (mm^2)	Yield Strength (MPa)	Mod. of Elasticity (MPa)
Rectangular	200 x100 x 5 150 x 90 x 3	2900 1404	17100 12096	360 320	229300 201000
Circular	165 x 4.7 110 x 1.9	2267 645	19016 8858	355 350	227000 220100

The columns were of different sizes, shapes, lengths and slenderness ratios. From the prototype sections of sizes (200 x 100 x 5) mm, (150 x 90 x 3) mm, (110 x 1.9) mm and (165 x 4.7) mm, three specimens of each section were prepared. One of these was filled with normal concrete; while the other was filled with lightweight concrete. End plates of 8mm thickness were welded to the column ends by 5mm fillet welds.

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 lightweight concrete. Details of the concrete mixes and material properties of the columns are summarized in Tables 2 and 3.

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 as shown in Figure 2.

DESIGN CONSIDERATIONS

The ultimate load-carrying capacity for a composite column can be calculated using several methods existing in codes of practice. The Bridge Code (BS 5400, 1979) and the Euro Code 4, 1985 contain rules for 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:

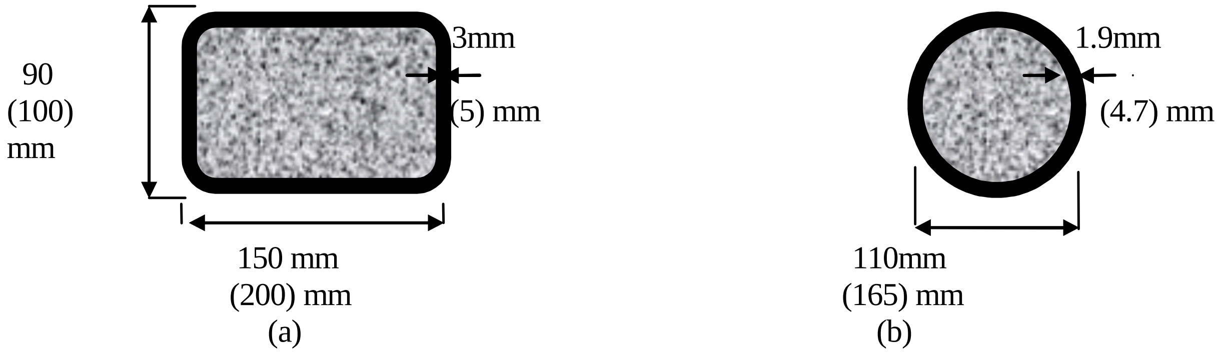

Figure 1: Cross-Sectional Dimensions for Test Specimens: (a) Concrete-Filled RHS; (b) Concrete-Filled CHS

Figure 2: Load Application on Column Specimen

The Bridge Code and Euro Code 4 for:

a -Rectangular or Square Sections are given as:

$$Nu = As\, fs\, k / \gamma_{ms} + Ac\, fck\, \gamma_{mc} \quad (1)$$

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 (fck) was taken as:

$$fck = 0.83\, fcu \quad (2a)$$

$$\textit{instead of} \quad fck = 0.67\, fcu \quad (2b)$$

where: fcu is the 28 day cube strength of concrete.

The value of 0.83fcu is recommended by EC4 for experimental work. Furthermore, the ratio between *Ac fck /*γ_{mc} and Ncu 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 *fsk* was taken as: *fsk* = 0.91fy.

b- Circular Sections: The squash load is given as:

$$Nu = 0.91\, As\, fy` + 0.45\, Ac\, fcc \quad (3)$$

in which, the enhanced concrete characteristic strength is:

$$fcc = c_1\, fyt / De + fcu,$$

and the reduced yield steel strength is:

$$fy` = c_2\, fy$$

where: c_1 and c_2 are constants depending on column length and its diameter. Also, the concrete contribution factor,

$$\alpha c = 0.45\, Ac\, fcc / Nu.$$

But, according to Euro code 4, the plastic resistance load is:

$$Nplrd = Aa\, fy / \gamma_a + Ac\, fck / \gamma_c.$$

In an axial loaded slender column, where the 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, Nd, is not greater than the uniaxial load (min. moment included in the design for slender columns due to imperfections), Ny, which is given by:

$$Ny = Nu\,[k_1y - \{k_1y - k_2y - 4k_3\}.\{My / M_{uy}\} - 4\,k_3\,\{My/M_{uy}\}2] \quad (4)$$

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 χ Nplrd, in which χ is a reduction factor due to the slenderness of the column.

Table 4. Designation and Results for Some Specimens

Col. Design- ation	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]	Design 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

Based on the rectangular full plastic stress distribution shown in Figure 3, the ultimate moment of resistance of a concrete filled rectangular hollow section can be calculated from the following equation (Hunaiti, 1997):

$$M_{uy} = fsk\,[0.5\,As\,(h` - dcy) + bt\,(t + dcy)] \quad (5)$$

where;

As: area of steel cross-section.

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:

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

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

$$\rho = fck / fsk. \quad (7)$$

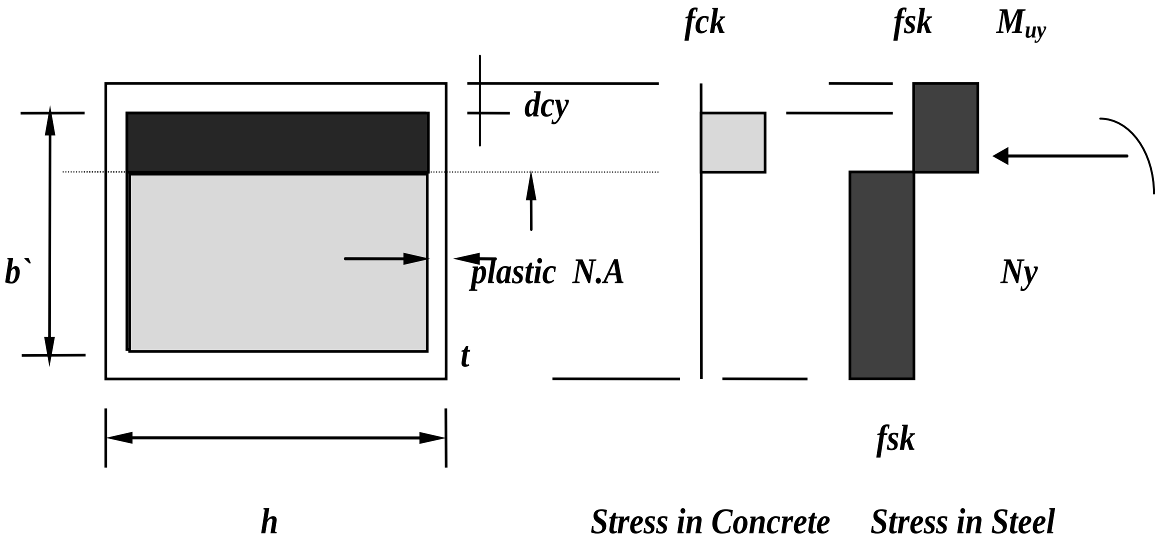

Figure 3: Stress Distribution in Concrete-Filled Rectangular Hollow Section at M_{uy}

Based on the rectangular full plastic stress distribution shown in Fig. 4, the ultimate moment of resistance of concrete filled Circular HS sections (in minor axis) can be calculated from the following equation:

$$M_{uy} = fsk\,.S\,(1 + 0.01m) \quad (8)$$

where *S*: the plastic section modulus of the composite column,

and *m* is given by:

$$m = (100/S)\,[t\,(De - t)\,2(\beta \sin\beta + \cos\beta - 1) + (1/4)\,\rho\,(De - St)\,3\omega] \quad (9)$$

*w*here:

$$\omega = 1/3 \cos 3\beta/4 - 1 \sin\beta\,(\pi - \sin 2\beta - 2\beta). \quad (10)$$

The depth of the neutral axis (or Cosine β) can be determined from the equilibrium conditions of the compressive and tensile forces, as defined by the stress distribution shown in Figure 4. Also, (m) can be determined using (BS 5400: Part 5) depending on "depth to thickness" ratio (De/t) and ρ [the ratio of stresses, which was defined before].

NUMERICAL RESULTS AND DISCUSSION

The behavior of column specimens under load is clearly indicaed in 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.

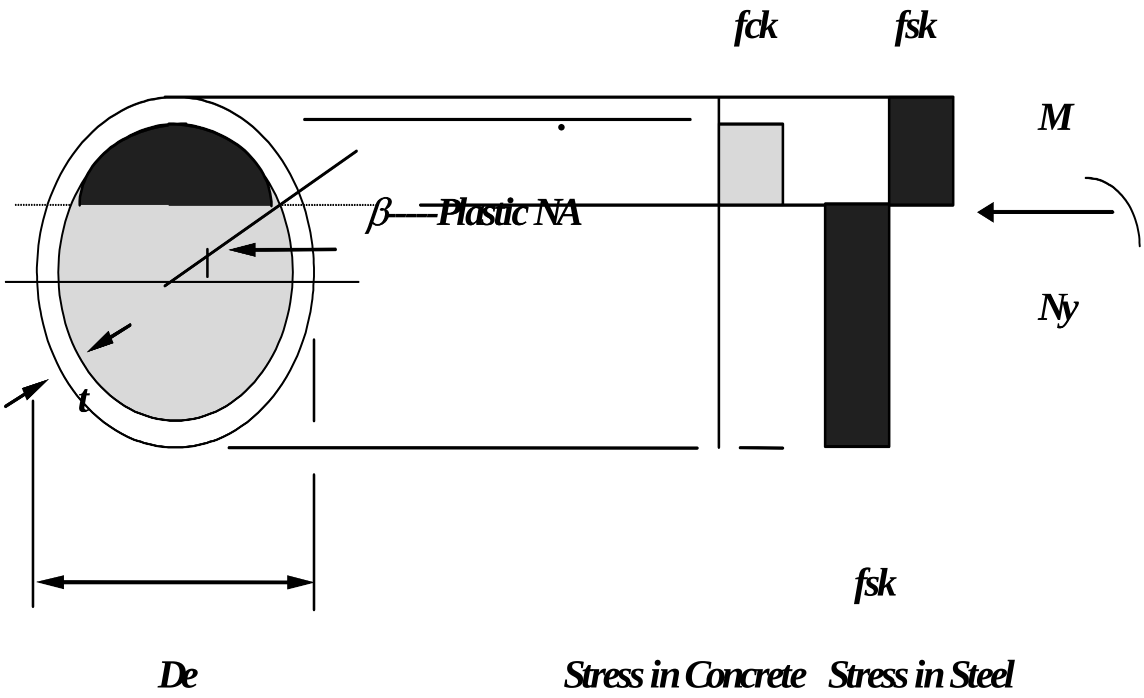

Figure 4: Stress Distribution in Concrete-Filled Circular Hollow Section at M_{uy}

The results of the tested columns are presented 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 and 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.

Figure 5: Mode of Failure for Some Tested Columns

Although both Eurocode 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 those of Eurocode 4. It can be obviously 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 normal concrete are more than three times stronger compared to those of lightweight concrete (cube strength of normal concrete is 33MPa; while it is 10MPa for lightweight concrete) (about 3.3 times greater), while a concrete contribution factor ratio, α, of about (2.89) showed an 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 section.

CONCLUSIONS

The steel tubes filled with lightweight aggregate concrete show acceptable strength under the applied load when compared to design calculations. According to the experimental and design code calculations, the behaviors of both lightweight concrete-filled steel tubular columns and normal concrete-filled steel tubular columns show a similar trend.

Columns filled with lightweight aggregate concrete exhibited local buckling. When the column reached failure load, an overall buckling took place as shown in Figure 5. 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. This exhibition can be seen from the results of comparisons between different types and dimensions of columns. 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 and thermal conductivity.

REFERENCES

British Standard Institution. 2000. BS 5400, Part 5, Concrete and Composite Bridges, U.K.

Commission of the European Communities. 1985. Eurocode 4, Common Unified Rules for Composite Steel and Concrete Structures, Brussles, Belgium.

Chapman, J. C. and Neogi, P. K. 1964. Research on Concrete-Filled Steel Tubular Columns, 1-26.

Ramamurthy, L. N. and Srinivasan, C. N. Salani and Sims, Chapman and Neogi, Gardner and Jacobson. 1978. Behavior of Concrete In-filled Tubular Columns, *J. of Instn. of Engrs.*, India, 58: 1-16 .

Sabalieish, A. 1988. The Feasibility of Producing and Developing Lightweight Concrete from Local Jordanian Raw Materials, Rep. (in Arabic), Royal Scientific Society (RSS), Amman-Jordan, 69-73.

Hunaiti, Y.M. 1997. Strength of Composite Sections with Foamed and Lightweight Aggregate Concrete, *ASCE-Journal of Materials in Civil Engg.,* 9 (2): 58-61.

Hunaiti, Y. M. 1996. Composite Action of Foamed and Lightweight Aggregate Concrete, *ASCE-Journal of Materials in Civil Engineering*, 8 (3): 111-113.

Brauns, J. 1998. Analysis of Stress State in Concrete-Filled Steel Columns. *Journal of Constructional Steel Research*, 49 (1999): 189-196.

Teng, J.G., Yao, J. and Zhao, Y. 2003. Distortional Buckling of Channel Beam-Column, 41: 595-617.

Zhang, W. and Shahrooz, B. M. 1999. Comparison between ACI and AISC for Concrete-Filled Tubular Columns, *Journal of Structural Eng*., 125 (11): 1213-1223.