Journal of Engineering Sciences, Assiut University, Vol. 40, No 3, pp.639-655, May 2012

THEORITICAL AND EXPERIMENTAL STUDY FOR THE BEHAVIOR OF SHORT COMPOSITE COLUMNS TESTED UNDER STATIC CENTRIC OR ECCENTRIC LOADS

L.M. Hafez[1], Y.R. Tawfic[2], A.B. Sdeek[3] and M. Abdelazeem[4]

1- Prof. Civil Eng. Dept., Minia University, Minia, Egypt.
2- Lecture. Civil Eng. Dept., Minia University, Minia, Egypt. (yasser_rifat@yahoo.com)
3- Lecture. Civil Eng. Dept., Bni Sweef University, Bni Sweef, Egypt, (amr_bakr2003@yahoo.com)
4- Demonstrator, Civil Eng. Dept., High Engineering & Technological institute, Minia, Egypt.(pyram2050@yahoo.com)

(Received November 2, 2011 Accepted February 28, 2012)

Steel concrete composite structures are widely used in many structural applications such as beams, slabs, walls and columns. Composite structures provide not only great reduction for the element size and weight but also high structural efficiency. Because of the little data and disagreement between the R.C. and steel Egyptian Codes of Practice, an experimental and theoretical investigation were conducted to study the behavior of concrete filled steel box columns. The experimental program consists of six square filled steel box columns and two square R.C. columns (reference elements). The experimental variables are the load position (centric or eccentric) and the distribution of the mechanical shear connectors (nails). Test results are presented and discussed in comparison with the estimated values predicated from the Egyptian R.C. and steel codes of practice as well as the values obtained from the finite element program (ANSYS 12).

KEYWORDS: *composite columns, steel plates, centric load, eccentric load, shear connectors*

1 INTRODUCTION

A steel-concrete composite column is a compression member, comprising either a concrete encased hot-rolled steel section or a concrete filled steel box of hot-rolled steel and is generally used as a load-bearing member in a composite framed structure. Steel-concrete composite columns are primarily used as columns supporting platforms in offshore structures, roofs of oil storage tanks, columns for large industrial workshops as well as piles and piers for bridges and viaducts. Because of their high resistance to earthquake, concrete filled steel box columns are widely used in multistory buildings, particularly in Japan.

Many theoretical and experimental researches [1, 2, 3, 4, 5, 6, 7, 8, 9, 10 and 13] were conducted to study the behavior of concrete filled steel box columns. However, many factors still need to be investigated such as bond between concrete and steel section, aspect ratio of concrete section, aspect ratio of steel plate section as well as the strength of the used concrete.

NOTATIONS

A_c	concrete core area	L_b	buckling length
A_r	area of longitudinal steel reinforcement	M_u	required flexural strength considering the second order effect
A_s	area of steel section	M_n	nominal flexural strength
c_1, c_2 and c_3	numerical coefficients taken as follows:- c_1=1.0, c_2=0.68 and c_3=0.40.	P_n	nominal compressive strength
		P_u	required compressive strength
e	eccentricity	r_m	radius of gyration of the steel shape
E_c	Young's modulus of concrete	x	neutral axis distance
E_m	modified Young's modulus $\geq E_s$	ε_{si}	strain in steel
E_s	Young's modulus of steel	ε_y	yield strain for steel
F_{cu}	concrete compressive strength	γ_c	safety factor for concrete
F_y	steel yield stress	γ_s	safety factor for steel
F_{ym}	modified yield stress	ϕ_c	strength reduction factor for compression members (0.8)
F_{yr}	yield stress of longitudinal steel reinforcement, t/cm^2	ϕ_c *and* ϕ_b	resistance factor for axial compression and flexural respectively

2 OBJECT AND SCOPE

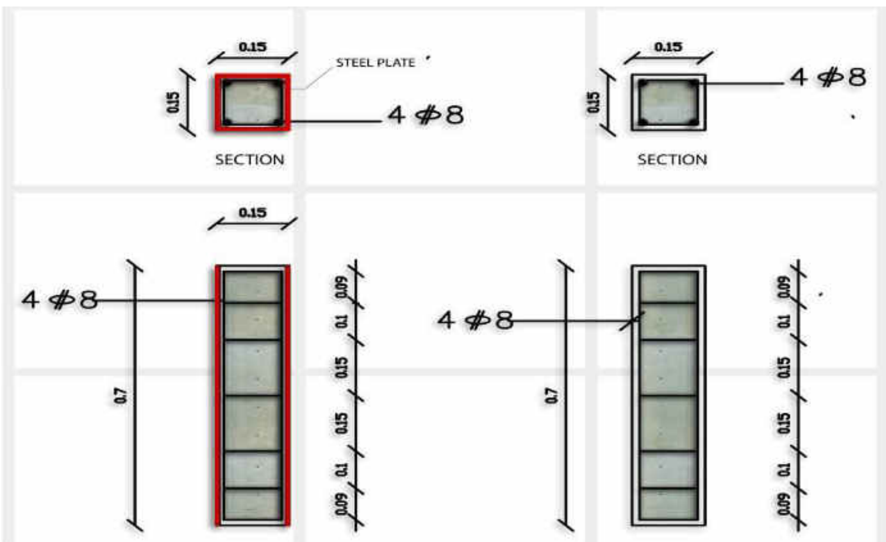
An experimentally investigation was carried out to study the behavior of concrete filled steel box column tested under centric or eccentric loading. A comparative theoretical study using a nonlinear finite element program (ANSYS 12) also conducted taking into account the aspect ratio of the overall cross section, the aspect ratio of steel plate and the value of load eccentricity. The main objective of this research work is to evaluate the existed knowledge and getting more information about the concrete filled steel box columns as well as to examine the applicability of the steel and reinforced concrete codes of practice [11 and 12] on such type of columns.

3 EXPERIMENTAL PROGRAM

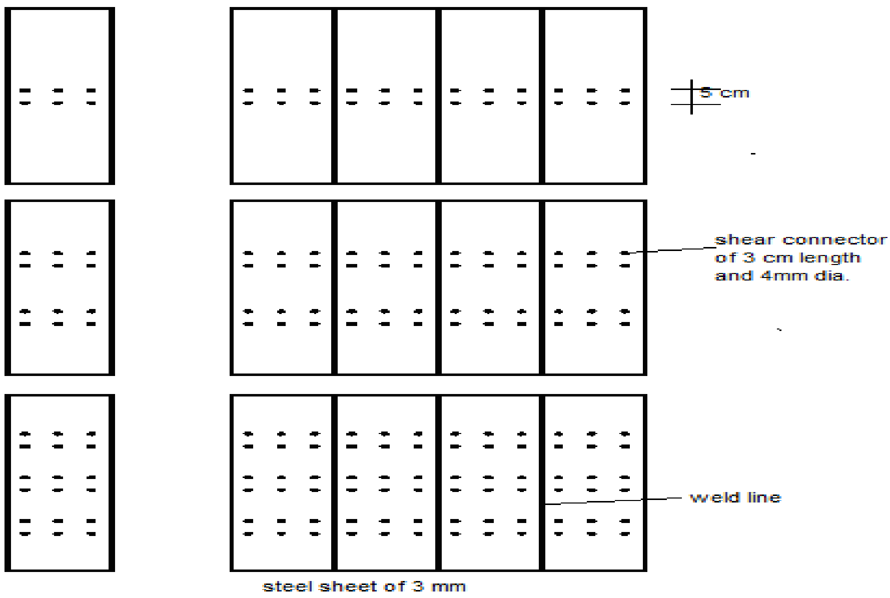
The experimental program consists of eight square short columns. Two reinforced concrete columns are cast as reference columns. Six columns are concrete filled steel box. Four steel plates (15 x 70 cm) (3 mm thickness) were drilled according to a certain arrangement to fix shear connectors and then welded together to make a steel box. The Box is filled with reinforced concrete to make the concrete filled steel box column. For all columns, the reinforced concrete section is 15x15 cm. The longitudinal reinforcements are four steel bars (8 mm diameter). Stirrups are made using four mm diameter steel bars. The experimental variables were the arrangement for the mechanical shear connectors and the type of load (centric or eccentric), see table (1) and figure (1).

Table (1): shows the details of the test specimens.

Category No.	Col. No	Type of column	Type of loading	Distribution of shear connector (3 cm length – 4 mm Diameter)
I	C_1	R.C (reference)	Static Centric load	-------
	C_2	composite		Two rows at the middle of height
	C_3			Four rows at the third of height
	C_4			Six rows at quarter of height
Π	C_5	R.C (reference)	Static Eccentric load (e/t=0.25)	-------
	C_6	composite		Two rows at the middle of height
	C_7			Four rows at the third of height
	C_8			Six rows at quarter of height



(a): Details of steel reinforcement.



(b): Arrangement of the shear connectors.
Figure (1) details of the test specimens.

3.1 Material properties

3.1.1 Steel bars and steel plates

For all columns, two different diameters mild steel bars were used as longitudinal reinforcements and stirrups. Tension tests were carried out for both the steel bars and the steel plates. Table (2) shows the mechanical properties for the steel bars and the steel plates.

Table (2): Mechanical properties of reinforcement steel

Steel	Actual diameter (mm)	Yield strength (Kg/cm^2)	ultimate strength (Kg/cm^2)	% Elongation
Diameter (8mm)	7.94	3280	4730	25%
Diameter (4mm)	3.8	2600	3500	---
Steel plate	thickness 3 mm	3100	3900	28 %

3.1.2 Concrete

Gavel and sand used for casting the test specimens were from local quires. The maximum nominal size for the gravel is 20 mm. The specific gravity for the gravel and sand are 2.53 and 2.63, respectively. The volume weight for the gravel and sand are 1.52 and 1.73 gm/cm^3, respectively.

Ordinary Portland cement produced by Assiut cement factory was used in this study, the specific gravity is 3.15. The surface area is 3200 cm^2/gm. The initial and final setting times are 2.25 and 4.25 hr, respectively.

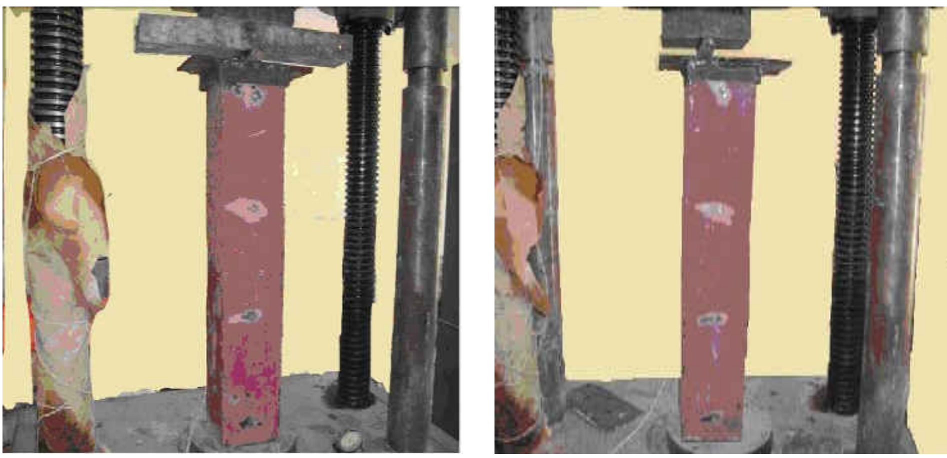
Table (3) presents the concrete mix proportions, which was designed to produce a 28 days compressive strength of about 250 kg/cm^2. The cement content was 350 kg/m^3, the fine to coarse aggregate ratio was two. Three concrete cylinders (Diameter = 15 cm and height = 30 cm), three prisms (10x10x50 cm), and six cubes (15x15x15 cm) were cast from every concrete mix. All concrete specimens were cured in a standard condition for 28 days and tested under static loading.

Table (3): concrete mix proportions.

Cement	**Water**		**Sand**	**Gravel**
(kg/m^3)	**($Liter/m^3$)**	**W/C**	**(kg/m^3)**	**(kg/m^3)**
350	175	0.5	400	800

4 TEST SETUP

The columns were tested under monotonically centric or eccentric load using 100 ton universal testing machine, see Fig. (2). Strains were measured both mechanically using extensometer and electrically using electrical strain gauges. The mid-height horizontal displacement was measured using dial gauge with an accuracy of 0.002 mm. During testing, the crack of the two control conventional columns only was marked due to invisible concrete surface of remaining composite columns. The failure mode was detected and recorded for al the specimens.



(a) Specimen tested under centric load (b) specimen tested under eccentric load**.**

Figure (2): Test setup.

5 TEST RESULTS

5.1 Modes of failure

5.1.1 Specimens of category I

Column C_1 (reference column) failed abruptly due to the formation of a longitudinal crack, then a sudden drop for the applied load was observed, see Fig (3-a). The failure was a brittle type. Higher values of ultimate load were recorded for the composite columns C_2, C_3, C_4 when compared with reference one. local buckling for the steel plate and crushing of concrete at the top or bottom side of the composite columns caused the failure of the composite columns (C_2, C_3, C_4), see Fig (3-b) through (3-d). It is worth to mention that the buckling of the steel plates was accompanied with a fracture for the welded line. Reaching the ultimate load for the composite column, a gradual descend for the applied load was observed producing a ductile type of failure that may be attributed to the high confinement effect for the steel box. The arrangement of the shear connectors showed no effect on the type of failure that may be due to the slenderness ratio of the tested columns (4.66) and the breadth to the thickness of steel plate (50). The geometry of the tested composite columns showed no clear effect on the mode of failure.

5.1.2 Specimens of category II.

The columns in category II were subjected to eccentric load (e/t = 0.25). Generally, the mode of failure for the specimens in category II showed no difference to the modes of failure for the specimens in category I. Specimen C_5 (reference column) had a brittle failure due to the sudden formation of longitudinal cracks; sees Fig (3-e). Similar to specimens C_2, C_3, and C_4, the composite columns (C_6, C_7, and C8) failed due to the local buckling for the steel plate and the crushing of concrete at the top or bottom side. Failure of welded line of the steel plate was also detected, see Fig ((3-f) - (3-g)).

(a) Specimen C_1

(b) Specimen C_2

(c) Specimen C_3

(d) Specimen C_4

(e) Specimen C_5

(f) Specimen C_6

(e) Specimen C_7

(f) Specimen C_8

Figure (3): Pattern of cracks and modes of failure for the test specimens.

Table (4): Experimental test results

Category	Column No	Cracking load (P_{cr}) (ton)	Local buckling load (P_{lb}) (ton)	Ultimate load (P_u) (ton)	P_{lb}/P_u	$P_u/P_{u\ (reference)}$
I	C1	24.0	--	26.5	---	1.0
	C2	Invisible	60.0	66.0	0.91	2.5
	C3	Invisible	62.0	72.0	0.86	2.72
	C4	Invisible	62.0	73.0	0.85	2.75
II	C5	12.5	--	13.5	---	1.0
	C6	Invisible	38.0	41.5	0.92	3.07
	C7	Invisible	38.0	43.5	0.87	3.22
	C8	Invisible	40.0	46.0	0.87	3.41

6 CRACKING, ULTIMATE, AND LOCAL BUCKLING LOADS

Table (4) presents the cracking load (p_{cr}), ultimate load (P_{ut}), and the visible local buckling load of the steel plates (P_{lb}) for all test specimens. Because of the steel box, the cracking load was not visible by eye so it could not be detected for all composite columns. However, the eccentricity of the applied load to the R.C column (C_5) resulted in about 52% of the cracking load of the reinforced concrete column (C_1). Compared to the reference column (C_1), composite columns C_2, C_3 and C_4 showed higher values of ultimate loads [1]. The percentage of increase for the values of ultimate load was 250%, 272%, and 275% for columns C_2, C_3, and C_4, respectively. It is clear that, the values of the ultimate load for the composite columns were enhanced due to the composite action between the steel box and the reinforced concrete core and due to the confinement of the concrete section created by the steel box. The concrete core is under tri-axial state of stress [2]. The inward buckling of the steel box is prevented by the concrete and the outward buckling of steel box is resisted by box action, bond between steel plates and concrete, and shear connectors. Increasing the number of the shear connectors rows from two to four resulted in 9 % increase for the values of the ultimate load. However, increasing the number of the shear connector's rows from four to six showed no significant effect on the values of the ultimate load for the composite columns which indicate a low sensitive to the distance between shear connectors at a certain limit. The local buckling load for the steel plates of the composite columns in category I ranged from 85% to 91% of the ultimate load.

Specimens in category (II) were subjected to eccentric load (e/t= 0.25). Similar to the specimens in category (I), composite columns showed higher values of ultimate loads when compared to the reference column (C_5). The percentage of increase for the values of ultimate load was 307%, 322% and 341% for columns C_6, C_7, and C_8, respectively. Compared with the reference specimens, the percentages of increase for the values of the ultimate load of the composite columns subjected to eccentric load was found to be higher than those of the composite columns subjected to centric load. The composite action is then highly advised for the columns subjected to eccentric loads. Increasing the number of the shear connectors rows from two to six resulted in 11% increase for the values of the ultimate loads. The local buckling load for the steel plates of the composite columns in category II ranged from 87% to 92% of the ultimate load.

7 Surface Strains

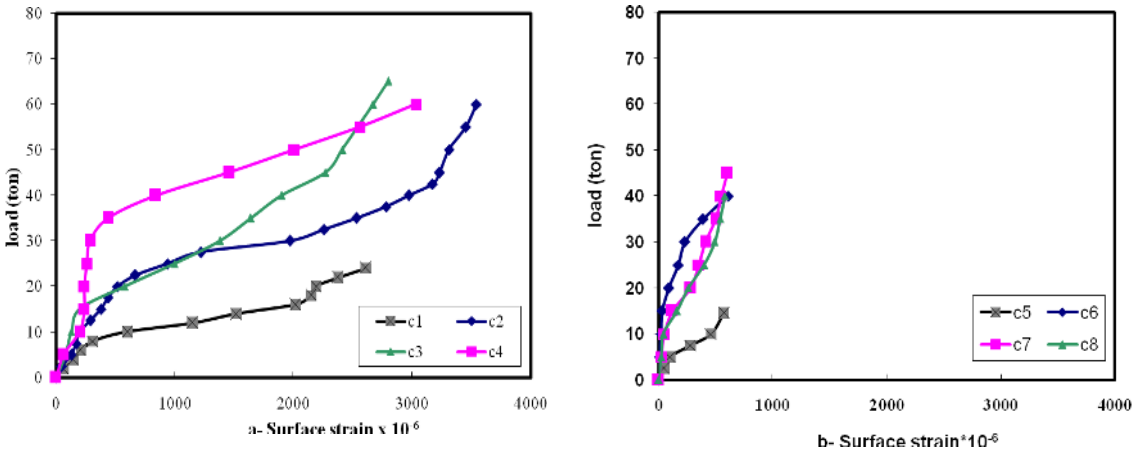


Figure (4): load-Surface strain relationship.

The values of longitudinal strain at the middle of the outside surface were measured using electrical strain gauges and mechanical extensometer for the composite columns and the reinforced concrete columns, respectively. Figure (4) presents the load-surface strain for the columns subjected to centric or eccentric loads. Generally, it was found that the values of stiffness of the composite columns are higher than those of the R.C columns. For columns subjected to centric or eccentric loads and at any certain load, the surface strains of the composite columns were lower than the surface strains of the R.C column. The ultimate surface strain for the reinforced concrete columns was found to be less than the compressive failure strain of concrete. Higher values of ultimate strain were recorded for columns subjected to centric loads than those of columns subjected to eccentric loads. It is thought that, the recorded values of strain for the columns subjected to eccentric loads were not the extreme values because the strain gauges were fixed on the front face of the columns and not on the side face that is highly affected due to the eccentricity.

8 HORIZONTAL DISPLACEMENT

Figure (5) shows the mid-height horizontal displacement for all test specimens in category I and category II. Generally, for columns subjected to centric or eccentric loads, the reinforced concrete columns showed higher values of mid-height horizontal displacement than the composite columns which is attributed to the lower rigidity of the reinforced concrete columns. Unexpectedly, the values of horizontal displacement at mid height of the composite columns subjected to eccentric loads were lower than those of the composite columns subjected to centric loads.

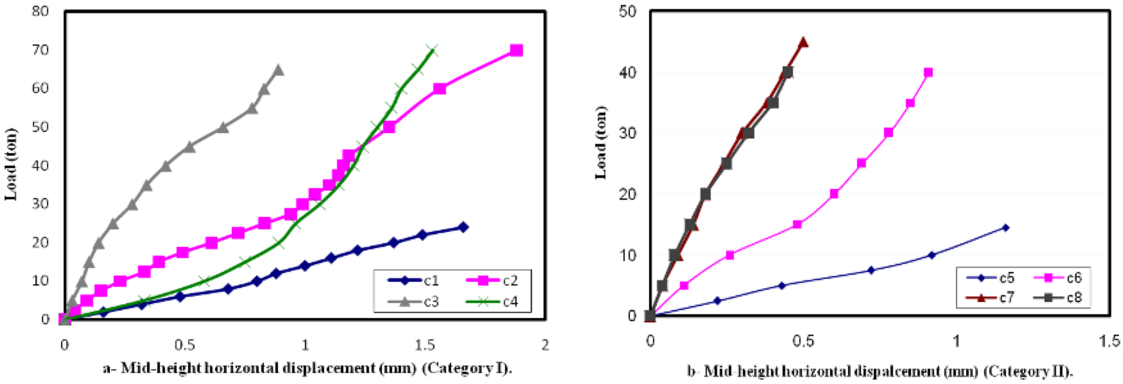


Figure (5): load- mid height horizontal displacement relationship.

9 THEORETICAL ANALYSIS

9.1 Finite Elements Idealization

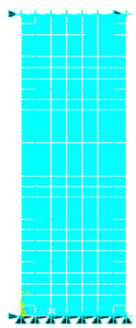


Figure (6): ANSYS program model.

In order to obtain more insight in to the behavior of the models, the models were analyzed using the soft ware package ANSYS12. Solid 65 elements were used to model the concrete. The rebar capability of this element was not considered. All steel bars and nails were modeled using link 8-3D spar element. Solid 45 elements were used for steel sheet. Contact elements 52 were used to contact the surface of the welded-formed steel sheet to the surface of the concrete. The bond between steel connector, welded-formed steel sheet and concrete was assumed as perfect. Full composite columns used for modeling. Avery fine mesh was provided to simulate the geometry of the analyzed models and to satisfy the requirement of used element's aspect ratio so the finite element model consisting of 1945, 1411, 1300 and 2500 numbers of solid 65, solid 45, link 8-3D spar and contacts, respectively . Figure (6) shows a typical full model for one of the analyzed models.

The total applied load was divided into a series of load increments. Newton Raphson equilibrium iterations provide convergence at the end of each load increment with to trance limits. The automatic time stepping in the ANSYS 12 program predicts and controls load step size for which maximum and minimum size are required. The finer increments were used to ensure that the buckling of steel sheet, crushing of concrete and failure loads could be predicted accurately. In this analysis the convergence was taken at 0.001 of displacement with maximum iteration number of 25 to reduce the accumulation forces within the iteration.

9.2 Formulas from Egyptian Codes of Practices

The Egyptian Codes for design and construction of concrete structures [11] and steel construction [12] developed two different formulas to predict the ultimate loads for composite columns.

9.2.1The EgyptianCodefordesignand constructionofconcretestructures [11]

Equation (1) is developed by the Egyptian Code for design and construction of concrete structures to design a composite column subjected to centric load.

$$P_u = 0.35\, f_{cu}\, A_c + 0.67\, f_r\, A_r + 0.67\, f_y\, A_s \qquad (1)$$

In case of eccentric load, the first principles which based on the concept of ultimate limit state is used to develop equation (2) and (3), see figure (7). The ultimate strength limit state for sections subjected to eccentric force should satisfy the equilibrium condition, compatibility of strain conditions, as well as the following consideration:

1- Strain distribution along the section are linear, therefore, the strains in the steel reinforcement and concrete are proportional to their distances from the neutral axis.
2- The resultant forces developed by the section must balance the applied loads for static equilibrium.
3- Concrete strength in tension is neglected, while steel reinforcement resists all tension stresses.
4- Maximum compression strains are taken 0.002

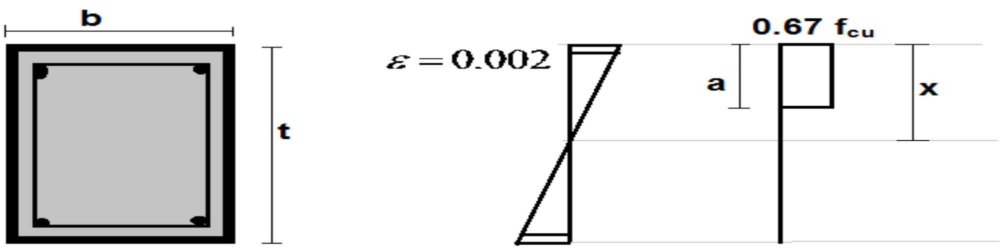


Figure (7): show first principle method

$$P_u = \frac{2}{3} \times \frac{f_{cu}}{\gamma_c} \times (0.8x)\, b + \sum_{i=1}^{i=n} A_{si}\, \varepsilon_{si}\, \frac{E_s}{\gamma_s} \qquad (2)$$

Where $\varepsilon_{si} = \dfrac{0.002}{x}\left(\varepsilon - d_i\right)$ If $\varepsilon_{si} > \varepsilon_y$ take $\varepsilon_{si} = \varepsilon_y$

$$P_u \times e = \frac{2}{3} \times \frac{f_{cu}}{\gamma_c} \times (0.8x)\, b \left(\frac{h}{2} - \frac{0.8x}{2}\right) + \sum_{i=1}^{i=n} A_{si}\, \varepsilon_{si}\, \frac{E_s}{\gamma_s}\left(\frac{h}{2} - d_i\right) \qquad (3)$$

9.2.2 The Egyptian Code for steel construction and bridges [12]

Equation (4) is developed by the Egyptian Code for steel construction and bridges to design a composite column subjected to centric load.

$$P_u = \phi_c \, p_n = \phi_c \, A_s \, F_{cr} \tag{4}$$

For inelastic buckling, $\lambda_m \leq 1.1$ $\quad F_{cr} = (1 - 0.348\, \lambda_m^{\,2})\, F_{ym}$

For elastic buckling, $\lambda_m \geq 1.1$ $\quad F_{cr} = 0.648\, F_{ym} / \lambda_m^{\,2}$

Where: $F_{ym} = F_y + c_1 \, F_{yr} \, (A_r / A_s) + c_2 \, F_{cu} \, (A_c / A_s)$, $E_m = E_s + c_3 \, E_c \, (A_c / A_s)$

λ_m = slender ratio = $L_b \, (F_{ym} / E_m)^{0.5} / \pi \, r_m$

Equation (5) and (6) are developed to design a composite column subjected to eccentric load.

For $P_u / (\phi_c \, p_n) \geq 0.2$

$$P_u / (\phi_c \, p_n) + (8/9) \{M_{ux} / (\phi_b \, M_{nx}) + M_{uy} / (\phi_b \, M_{ny})\} \leq 1.0 \tag{5}$$

For $P_u / (\phi_c \, p_n)$ 0.2

$$P_u / (2\phi_c \, p_n) + \{M_{ux} / (\phi_b \, M_{nx}) + M_{uy} / (\phi_b \, M_{ny})\} \leq 1.0 \tag{6}$$

9.3 Theoretical results and discussion

9.3.1 Modes of failure

Similar to the experimental test results, the theoretical analysis using ANSYS12 showed a formation of local buckling for the steel plate near the top head followed by the final failure of the composite columns, see Fig. (8).

a. Composite column under centric load. b. Composite column under eccentric load.

Figure (8): Failure of composite columns using ANSYS12 program.

9.3.2 Cracking, local buckling and ultimate Loads

Table (5) presents the theoretical (ANSYS12) values of cracking load ($P_{cr\text{-}th}$), local buckling load ($P_{lb\text{-}th}$), and ultimate load ($P_{u\text{-}th}$). The ultimate values of load given by the Egyptian Code for design and construction of concrete structures ($P_{u\text{-}cc}$) and the ultimate values of load given by the Egyptian Code for steel construction and bridges ($P_{u\text{-}sc}$) are also presented in Table (5). The experimental and theoretical results showed clearly that the eccentricity of the applied load (e/t=0.25) highly affected the values of cracking, local buckling and ultimate loads for the reinforced concrete and the composite columns. The values of ultimate load for the R.C column subjected to eccentric load was about 51% of the value of the ultimate load for R.C columns subjected to centric load. In case of the composite columns, the percentage of ultimate load for columns subjected to eccentric load to the ultimate load for columns subjected to centric load ranges from 60% to 66%. Hence, the composite columns provide better behavior under eccentric load than the R.C columns.

Table (5) Theoretical and experimental test results

Category	Column No	Theoretical Load by Ansys12 (ton)			P_{u-exp}	P_{cr-th} / P_{u-th}	P_{lb-th} / P_{u-th}	P_{u-th} / P_{u-exp}	$\frac{p_{u-sc}}{p_{u-\exp.}}$	$\frac{p_{u-cc}}{p_{u-\exp.}}$
		P_{cr-th}	P_{lb-th}	P_{u-th}						
I	C1	22	---	31.5	26.5	0.70	---	1.19	---	0.87
	C2	65	68	76.5	66	0.85	0.89	1.16	1.03	0.79
	C3	66	71	77	72	0.86	0.92	1.07	0.94	0.73
	C4	66	72	77	73	0.86	0.94	1.05	0.93	0.72
II	C5	11	---	16.8	13.5	0.65	---	1.24	---	1.04
	C6	37	39	45.5	41.5	0.81	0.86	1.10	0.90	0.77
	C7	38	40.6	46.6	43.5	0.82	0.87	1.07	0.85	0.74
	C8	41.5	45	50.5	46	0.82	0.89	1.1	0.80	0.7

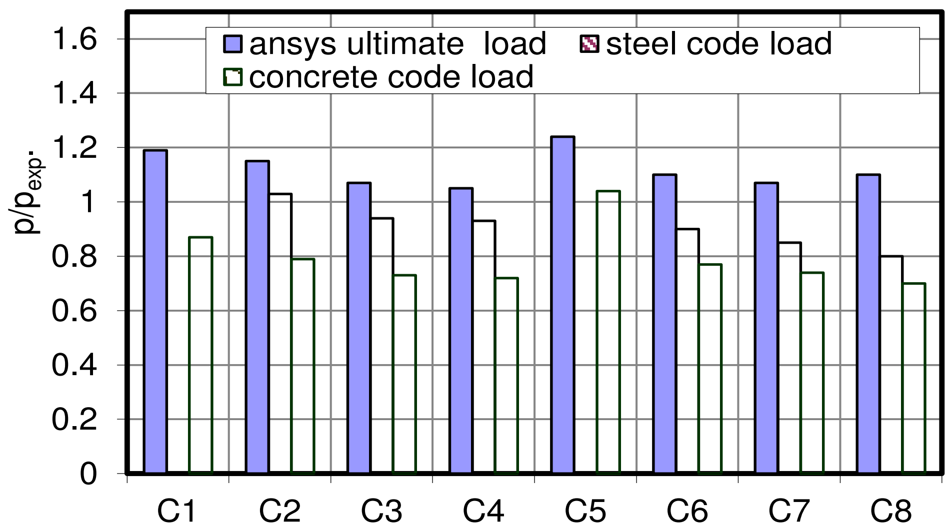


Figure (9): The ratios of ultimate load for the theoretical and experimental results.

Table (5) shows also that the presence of shear connectors has higher effect on the values of the cracking, local buckling and ultimate loads for the composite columns subjected to eccentric load than the case of composite columns subjected to centric load. The ratio of theoretical cracking load of composite columns to the theoretical ultimate load slightly affected by the eccentricity of the applied load. The average ratios of theoretical cracking load to the ultimate load were 0.86 and 0.82 for centric and eccentric load respectively. The average theoretical value for the ratio of the local buckling load to the ultimate load was found to be about 0.9. This ratio is slightly higher (about 5%) for composite columns subjected to centric load than those of composite columns subjected to eccentric load.

From the theoretical values calculated from Equations (1) through (6), the finite element analysis results, and the experimental test results shown in Table (5) and Fig. (9), it could be implies that:

- The Egyptian Code for design and construction of concrete structures under estimates the ultimate capacity of the composite columns subjected to centric or eccentric load by about 21% ~ 30%. The E.C.P equation (2) assumes a uni-axial strength for the concrete although the existence of a steel box has a confinement effect on the inside concrete resulting in a tri-axial state of stress.
- The Egyptian Code for steel construction and bridges showed a conservative estimation for the values of the ultimate load.
- The finite element method through ANSYS program over estimates the ultimate load capacity by (15%, 7% and 5%) for columns C2, C3 and C4, respectively. It also over estimates the ultimate load capacity by (10%, 7% and 10%) for C6, C7and C8, respectively. These percentages of increase may be due to the rough convergence tolerance or small number of iteration.

9.3.3 Surface strain

The experimental longitudinal surface strain is plotted against the theoretical results at the same places in Fig. (10) and Fig. (11). The curves show a good agreement between the theoretical and experimental test results.

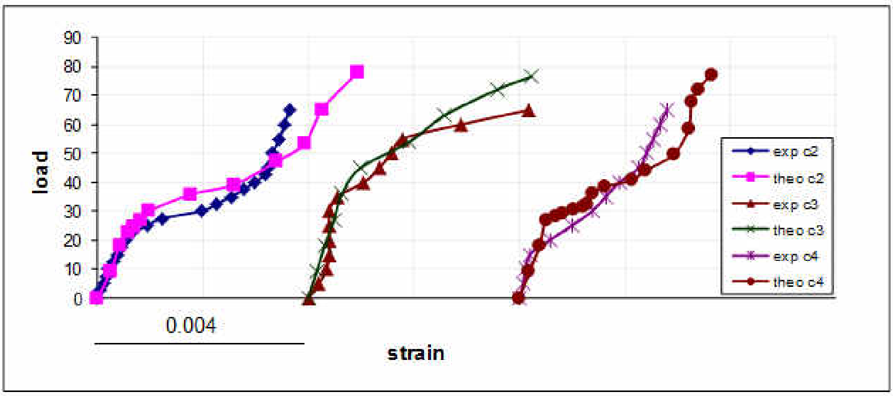


Figure (10): load – surface strain relationships for the tested specimen in category I.

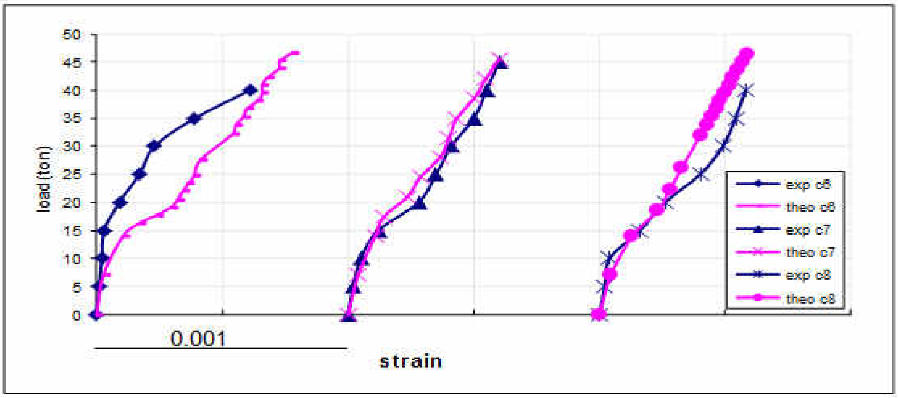


Figure (11): load – surface strain relationships for the tested specimen in category II.

10 CONCLUSIONS

Based on the theoretical and experimental studies, the following can be concluded:

1- The concrete filled steel box showed a relatively ductile failure due to the confinement effect of the steel box.

2- The concrete filled steel box tested under centric loading showed an improvement for the ultimate load of about 250-275 %, meanwhile, composite columns tested under eccentric loading produced an improvement of about 307-341%. This improvement can be attributed to the effect of the confinement action in composite columns, as well as the increase of the total area of steal used.

3- The detected longitudinal surface strain showed higher values of stiffness for the composite columns than the R.C columns.

4- The Egyptian Code of practice (E.C.P) for concrete structures [11] underestimates the ultimate load of the composite columns subjected to either centric or eccentric loading by about 21% ~ 30%. The E.C.P considers a uni-axial state of stress for the concrete core of the composite columns. However, the actual state of stress of these columns is a tri-axial one.

5- The Egyptian Code for steel construction and bridges [12] showed a conservative estimation for the values of the ultimate load.

6- The finite element method through the application of ANSYS program over estimates the ultimate load capacity for composite columns by an average value of 10 %. These percentages of increase may be due to the rough convergence tolerance or small number of iteration.

7- The average ratio between the crack and ultimate theoretical loads for composite columns (P_{cr-th} / P_{u-th}) was found to be equals to 0.83 and more or less has not be affected by the eccentricity. In case of R.C columns, this ratio was found to be about 0.67 because of the lower exhibits confinement with compared with composite columns.

8- The average theoretical value for the ratio of the local buckling load to the ultimate load was found to be about 0.9. This ratio is slightly higher (about 5%) for composite columns subjected to centric load than those of composite columns subjected to eccentric load.

REFERENCES

1. Ge, H., and Usami, T. (1992) "Strength of Concrete-Filled Thin-Walled Steel Box Columns: Experiment", Journal of Structural Engineering, ASCE, Vol. 114, No. 11, Nov., pp. 3036-3054.
2. Shams, m., and Saadeghvaziri, M. A. (1999) "Nonlinear Response of Concrete Filled Steel Tubular Columns Under Axial Loading", ACI, Strutural Journal, Vol. 96, No. 5, Nov.-Dec., pp. 1009-1017.
3. Yagishita F., Kitoh H., Sugimoto M., Tanihira T. and Sonoda K. (2000). “Double skin composite tubular columns subjected to cyclic horizontal force and constant axial force”, In: Proceedings of the Sixth International Conference on Steel and Concrete Composite Structures, USA, 497–503.
4. O'Shea, M. D., and Bridge, R. O. (2000) "Design of Circular Thin Walled Concrete Filled Steel Tubes", Journal of Structural Engineering, ASCE, Vol. 126, No. 11, pp. 1295-1303
5. Shakir-Khalil, H., and Zeghiche, J. (1998) "Experimental Behavior of Concrete-Filled Rolled Rectangular Hollow-Section Columns", The Structural Engineering, Vol. 67, No.19, Oct., pp. 346-353.
6. Amit H. Varma; James M. Ricles; Sause and Le-Wu Lu (2002) "Experimental Behavior of High Strength Square Concrete-Filled Steel Tube Beam-Columns", Journal of Structural Engineering, ASCE, Vol. 128, No.3, March., pp. 309-317.
7. Lin M.L. and Tsai K.C. (2001). “Behavior of double skinned composite steel tubular columns subjected to combined axial and flexural loads”, In: First International Conference on Steel & Composite Structures, Pusan, Korea, 1145–1152.
8. Euro code 4. (2005). “Design of steel and concrete structures, Part1.1, general rules and rules for building”, Brussels (Belgium): European Committee for Standardization.
9. Han L.H. (2007). “Concrete filled steel tubular structures (Second edition)”, Beijing: China Science Press. [In Chinese].
10. Han L.H., Huang H., Tao Z. and Zhao X.L. (2006). “Concrete-filled double skin steel tubular (CFDST) beam-columns subjected to cyclic bending”, Engineering Structures, Vol.28, No.12, 1698–714.
11. Egyptian code of practice for design of concrete structure, ECP 203-2006.
12. Egyptian code of practice for steel construction and bridges, ministerial decree 359-2007.
13. Tao Z. and Han L.H. (2006) “Behavior of concrete filled double skin rectangular steel tubular beam -columns”, Journal of Constructional Steel Research, Vol.62, No.7, 631-646.
14. Han L.H., Huang H. and Zhao X.L. (2009a). “Analytical behavior of concrete-filled double skin steel tubular (CFDST) beam-columns under cyclic loading”, Thin-Walled Structures, Vol.47, No.6-7, 668-680.

دراسة عملية ونظرية لسلوك الاعمدة المركبة القصيرة المختبرة تحت تأثير الاحمال الاستاتيكية المركزية واللامركزية

انتشراستخدام الاعمدة المركبة من الصلب المفرغ المملوءة بالخرسانة المسلحة فى العديد من التطبيقات الهنسية مثل الكمرات والبلاطات والاعمدة والحوائط وذلك لكفائتها العالية وتقليل القطاعات الانشائية. فان الهدف من البحث معرفة سلوك الاعمدة المركبة المختبرة تحت أحمال محورية وغير محورية وبمقارنتها ببرنامج عدد لانهائى من العناصر وكذلك معادلة كود الخرسانة والصلب المصرى. فقد احتوى البرنامج العملى على ثمانية اعمدة منقسمين الى مجموعتين. المجموعة الاولى اختبرت تحت حمل محورى والثانية تحت حمل غير محورى (نسبة اللامركزية الى سمك القطاع = 0.25). كل مجموعة مكونة من اربعة اعمدة واحد منها قطاع خرسانى فقط ذات أبعاد 15*15 سم ومسلح 4 أسياخ قطر 8 مم والثلاثة الاخرى قطاعات مركبة ذات سمك اللوح الصلب 3 مم وذات توزيع لمسامير الربط مختلفة (أما التوزيع فى منتصف العمود أو ثلثه أو ربعه). وتبين من الاختبارات أن الاعمدة المركبة من الصلب المفرغ والمملوْة بالخرسانة المسلحة تعطى أحمال انهيار أعلى بمقدار حوالى 2.7 مرة من عمود الخرسانة وذلك فى حالة المجموعة الاولى أما فى حالة المجموعة الثانية فبمقدار 3.3 مرة من عمود الخرسانة فقط وذلك يدل على مقاومة الاعمدة المركبة للاحمال اللامركزية. وتبين ان الاحمال الانهيار لا تتأثر تأثير كبير مع تغير مسامير الربط وذلك لصغر طول العمود المختبر. وتم مقارنة نتائج العملى بالبرنامج النظرى وجد انه يوجد تطابق ملحوظ مع الاختبارات العملية وان نسبة حمل الانهيار الى حمل الانهيارالخاص بالعملى حوالى 1.1 . وعند مقارنة الاختبارات العملية بكود الخرسانة تبين ان النسبة كانت تقريبا 0.75 فى حالة الحمل المحورى و الغير محورى وذلك لان كود الخرسانة يتجاهل أن الخرسانة مقيدة . اما مع مقارنة العملى مع كود الصلب تبين ان النسبة 0.95 فى حالة الحمل المحورى و 0.85 فى حالة الحمل الغير محورى.