AXIAL STRENGTH OF NORMAL AND HIGH STRENGTH CONCRETE-FILLED STEEL BOX COLUMNS

M. Nasser Darwish and Tarek I. Ebeido
Structural Engineering Department, Faculty of Engineering,
Alexandria University, Alexandria, Egypt

This paper presents an experimental study on the behaviour of short concrete-filled square steel box/tube (CFT) columns loaded in compression to failure. Twenty-Six column specimens (150x150x1000mm) were tested under concentric and eccentric compressive loads, to investigate the effects of several variables on the axial strength and behaviour of the composite columns. The main parameters studied include: using both normal strength concrete (28 MPa, 280 kg/cm^2) and high strength concrete, HSC (62 MPa, 620 kg/cm^2, 8.9 ksi); the presence of steel studs welded to the inner faces of the steel box; and varying the steel tube thickness (t= 2.5, 4.5 and 6.0 mm). Besides, the inclusion of longitudinal steel reinforcement and transverse stirrups were also studied. For the tested columns longitudinal and transverse strains were measured; load-strain relationships recorded; ultimate strength and collapse modes monitored; and the structural behaviour investigated. Test results revealed the significant effect of using high strength concrete; the unpronounced confinement produced by square tubes; and the marginal effect of welded studs on increasing the axial strength of CFT columns. Axial capacity test results were compared to theoretical predictions of some current codes, e.g. ACI (95) and Eurocode4 (EC4-96), to assess such codes' equations. Close agreement was found with ACI predictions for the studied cases; in contrast to EuroCode4 that overpredicts axial strength. Hence, it seems adequate to calculate the axial strength of short square CFT columns based on adding the resistances of its components as recommended by ACI. Results are presented and other conclusions drawn.

يقدم هذا البحث دراسة معملية عن سلوك الأعمدة الحديدية المربعة المملوئة بالخرسانة (Concrete Filled Steel Columns) والمعرضة لقوى ضغط حتى الانهيار. وقد تم إختيار ٢٦ عامود (١٥٠X ١٥٠X ١٠٠٠مم) تحت تأثير أحمال ضغط محورية ولامحورية، لدراسة تأثير العوامل المختلفة على مقاومة وسلوك هذه الأعمدة المركبة. وشملت المتغيرات الأساسية: إستخدام خرسانة عادية المقاومة (٢٨ ميجا باسكال، ٢٨٠ كجم/سم٢) وخرسانة عالية المقاومة (٦٢ ميجا باسكال، ٦٢٠ كجم/سم٢) ، ووجود أربطة قص (Connectors) ملحومة بالسطح الداخلى للغلاف الخارجى ودراسة تأثير تغير سمك الغلاف الحديدى الخارجى (٢,٥،٤,٥،٦,٠ مم). بالاضافة الى هذا فانه قد تم دراسة تأثير وجود حديد تسليح طولى وكانات عرضية. وقد تم قياس الإنفعال الطولى والعرضى للأعمدة المختبرة وايجاد العلاقة بين الحمل والإنفعال وقياس حمل الكسر وتتبع شكل الانهيار ودراسة السلوك الإنشائي للأعمدة المختبرة. وقد أظهرت النتائج المعملية التأثير الفعال لإستخدام خرسانة عالية المقاومة فى حين أن إستخدام أربطة القص أعطى تأثيرا هامشيا على مقاومة الأعمدة المركبة المختبرة وكذا لم يظهر أثرا مذكورا لوجود تقييد جانبى للخرسانة (Confinement) . ود تم مقارنة النتائج المعملية مع معادلات بعض المواصفات الأمريكية والأوروبية. وقد أظهرت المقارنة أن معهد الخرسانة الأمريكى الدولى (ACI) يعطى نتائج قريبة من النتائج المعملية فى حين أعطت المواصفات الأوروبية (Euro Code 4) نتائج أعلى من النتائج المعملية. ومن ثم يتبين انه يمكن تقدير المقاومة المحورية لهذه النوعية من الأعمدة المركبة بجمع مقاومة مكونات عناصرها من خرسانة وحديد كما هو متبع فى مواصفات معهد الخرسانة الأمريكى الدولى (ACI).

Keywords: axial Strength, Composite columns, Concrete-filled steel tubes, High strength concrete, Reinforced concrete.

INTRODUCTION

The use of concrete filled steel hollow tubes (CFT) columns has been widely spread in numerous civil engineering applications throughout the world, e.g., as columns in multistory buildings in platforms supporting offshore structures; in piles and piers roofs of oil storage tanks ...etc [1-3]. These may be circular or rectangular sections filled with concrete to form a composite column. The advantages of such type of columns are the enhanced properties due to the composite action of its constituent materials. These can be summarized as follows: (i) large stiffness, high strength and high ductility; (ii) large energy absorption capacity; (iii) high fire resistance; (iv) restraint to local buckling of the steel tube provided by the filled concrete and hence stability and greater buckling load; (v) increase in concrete strength due to the confining effect provided by the steel tube; (vi) omission of formwork leading to a reduction in construction cost and time; and

(vii) excellent axial and flexural load carrying capacity, and high shear resistance (4-5]. Hence, CFT columns increase load carrying capacity for a reduced cross section and have good earthquake resistant properties and therefore are ideal to be used in areas subject to seismic loading [6].

Numerous experimental and theoretical investigations were conducted throughout the years to study the behaviour of concrete filled steel box columns. Shams *et al.* [5] mentioned that Kloppel and Goder established lower and upper limits to predict the strength of concentrically loaded CFT columns. Gardner and Jacobson [7] suggested that as the steel tube restrains the concrete core at failure, an internal pressure develops between the steel tube and concrete creating a tensile hoop stress in the steel tube and due to this confining effect, the compressive strength of concrete will be augmented. Furlong [8] proposed that the lower limit of the axial capacity of CFT columns could be established as the force necessary to cause the steel to yield plus the force which is required to develop the same strain in the concrete. Neogi and San [9] developed a numerical procedure to study the load-deformation behaviour of CFT columns over the elasto-plastic range. Knowles and Park [10] developed a method for calculating the limits of slenderness ratios that determine whether an increase in concrete strength due to triaxial confinement is likely or not. Tomii and Yashimaro 1977 (cited in Shams *et al.* [5]), concluded that CFT long columns can fail in a general buckling mode whereas the mode of failure in the case of shorter columns will be crushing of concrete. Ge and Usami [11] concluded that the stiffening effect of longitudinal stiffeners is significant in the case of CFT columns. Bridge and Webb [12] concluded that local buckling occurred almost at the same level of axial load in both hollow and filled columns and therefore the use of high strength concrete in such columns might be questionable. Rangan and Joyce [13] used an iterative technique to design slender columns, assuming that the axial capacity of a slender eccentrically loaded CFT column was reached when the maximum moment was equal to the ultimate bending moment at the midheight of the column. Zein El-Din *et al.* [14,15] developed a computer program based on elasto-plastic analysis for predicting the ultimate load and the relation between load and corresponding lateral displacement. Hajjar and Gourley [16] developed a polynomial equation, similar to that used for conventional reinforced concrete, to represent the three-dimensional cross-section strength of square or rectangular CFT beam columns. Bradford [17] presented a method with close approximation to the failure envelope of short CFT rectangular columns, ignoring the effects of steel local buckling and concrete creep and shrinkage. Schneider [6] studied axially loaded short concrete filled steel tubes concentrically loaded in compression and reported test results of several circular, square and rectangular specimens.

Recently, high strength concrete, HSC, (say concrete with cylinder strength, $f_c' > 40$ MPa) is being widely used in the construction industry. However, their use in CFT columns has lagged behind [5]. Boyd *et al.* [18], studied circular encased concrete columns with shear studs attached to the interior surface of the shell under reversed cyclic lateral loads, concluding that high strength concrete in the column core resulted in higher ultimate strength but greater strength degradation and lower energy dissipation when compared to columns with normal strength concrete cores. Shams and Saadeghvaziri [5] recommended studying the effect of using high strength concrete, HSC, on the ultimate strength of concrete filled steel tubular columns, since there was uncertainty about the order of occurrence of local buckling of the steel tube and crushing of the concrete core. Besides, they recommended investigating the effect of using steel dowels (studs) welded to the inner face of the steel tube on improving the bond between concrete and steel. Zhang *et al.* [3] studied both short and slender rectangular normal and high strength steel tubes filled with normal and high strength concrete and proposed revisions to American Concrete Institute (ACI) procedures. Kilpatrick *et al.*

[2] experimentally studied eccentrically loaded CFT columns under both single and double curvature with high strength materials for both concrete core and steel shell. Hooper *et al.* [1] reported practical applications on the use of HSC in CFT making full use of the best of both materials. Other issues in CFT studies include local buckling, cyclic loading, fire resistance and more important long term effects (i.e., creep, shrinkage) which are essential in concrete construction. Bradford [17] cited investigations on fire resistance; high strength materials; shear capacity; and time effects. Besides, as only small-scale test specimens were mainly used in tests, hence when results of such investigations are extrapolated to represent full-scale large columns, problems might arise as the effects of steel shell contribution and concrete confinement may diminish.

DIFFERENCE BETWEEN CONCRETE-FILLED TUBES AND CONVENTIONAL COLUMNS

Steel tubes in CFT columns can be used as longitudinal and confining (transverse) reinforcement, besides, confinement is continuous, unlike conventional reinforced concrete columns. These are some favorable characteristics for CFT to be used in regions of high seismic risks. Moreover, there are considerable differences between CFT and conventional reinforced concrete columns under sustained loading [5, 6]. The concrete coefficient of contraction in CFT is low and shrinkage proceeds slowly due to humid environment inside the steel tube, and concrete expands more than its steel jacket under large longitudinal strain. Contraction of the concrete hardly affects the load carrying capacity. While in conventional reinforced concrete columns, concrete experience contraction, by a lengthy period of shrinkage and creep under loads. Besides, there are differences in strength of concrete cured within a CFT versus concrete cured for cylinder tests.

The Behaviour of Concrete-Filled Tubes

The behaviour of CFT columns depends on whether both the steel tube and concrete are loaded simultaneously in compression; or whether the load is applied exclusively to the concrete core; or whether the steel tube is loaded [4-6]. If CFT is loaded simultaneously in compression, the steel tube expands more than the concrete core under moderate loads, since Poisson's ratio is higher for steel sections, suggesting that the steel tube offer the concrete core no or little confinement at early loading stages. With increasing longitudinal strain, and once concrete begins to crush [16], the lateral expansion of the unconfined concrete gradually becomes greater than that of steel tube, and the concrete Poisson's ratio increases considerably that concrete expands and increases in volume due to micro-cracking. Hence internal pressure develops at the steel tube-concrete interface, and the steel tube is engaged and there is transfer of load from the tube and the concrete core. Concrete is stressed nearly triaxially and the tube biaxially. This induces concrete confinement by the steel tube, hence increasing the overall load resisting capacity of CFT columns. However, as previously reported, this was mostly noted for circular tubes rather than for rectangular ones, and for short columns only [3,5,6]. The flat sides of square CFT provide less confining restraint than circular CFT, because the wall of the square tube resists concrete pressure by plate bending instead of membrane–type hoop stresses [5,16]. Hence concrete in square and tubes generally may undergo little increase in strength due to confinement. Sometimes the effect of confinement in square tubes is assumed to produce only an increase in ductility and no axial load increase [16]. This has called upon avoiding square and rectangular CFT for their low ductility in regions of moderate and high seismic risks. Besides, for columns with large ratio of length/side dimensions the composite section may fail by column buckling before reaching the strains necessary to cause an increase in the concrete core volume and hence utilization of composite action [6]. The behaviour of CFT depends mainly on: i) the ratio of tube width to tube thickness; ii) the relative strengths of the steel and concrete, ratio of concrete compression strength to steel yield strength; f'_c/f_y ; and iii) slenderness ratio; cross sectional shape; aspect ratio; and eccentricity.

Required Research

From the available literature it can be seen that concrete core confinement in CFT columns is not well predicted. Besides, confinement was only observed at high axial load levels. Moreover, there is lack of information on noncircular tube columns as these may provide no confinement; and on the behaviour of eccentrically loaded columns. Furthermore, the effects of utilizing high strength concrete, HSC, in CFT columns are rather questionable and not fully understood since unconfined HSC tend to show brittle failure. Besides, improvements of current code provisions are necessary to provide rational design basis for steel-encased concrete members and to realize the benefits of such type of composite construction.

The Current Research

In the current paper the effects of using high strength concrete on the axial strength of short square concrete filled steel tube, CFT, columns as well as the effect of using steel studs (dowels) welded to the inner face of the steel tube on improving the bond between concrete and steel are studied. This is performed through an experimental investigation including 26 columns. Sixteen columns are concrete filled steel tubular square columns; eight columns are reinforced concrete columns; and two columns are reinforced concrete filled steel columns. The effect of other parameters are also studied such as: (i) width to wall thickness ratio of the steel tube; (ii) effect of load position whether concentric or eccentric; and, (iii) effect of including reinforced concrete for the concrete filled steel columns instead of plain concrete. In addition, the behaviour, axial strength, and failure mode of concrete filled steel columns and reinforced concrete columns are compared for both normal and high strength concrete. For the tested columns, longitudinal and transverse strains were traced at the columns' mid-heights and hence load-strain relationships were determined. Finally, the validity of the equations presented in some building codes, e.g. Eurocode 4 (EC4) [19] and ACI 318-95 [20], for the estimation of the ultimate strength of concrete filled steel columns are assessed.

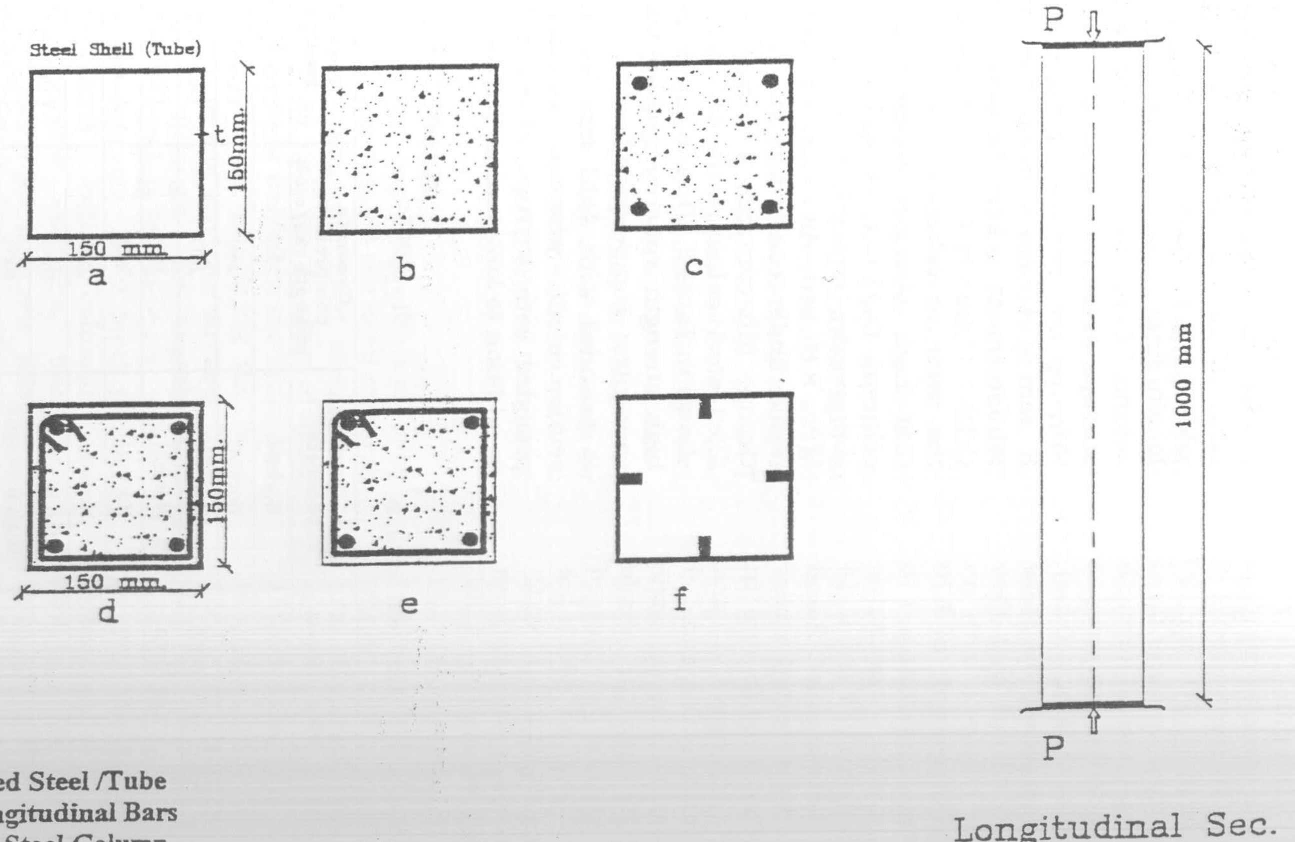
EXPERIMENTAL PROGRAM

An experimental program was undertaken to study the behaviour, axial strength, and failure mode of concrete filled steel square box columns. The main objectives of the experimental program are: (i) to study the effect of using high strength concrete (HSC) on the ultimate strength of concrete filled steel columns; (ii) to investigate if using steel dowels (studs) welded to the inner faces of the steel tube affects the load carrying capacity of CFT columns; (iii) to study the effect of width to wall thickness ratio of the steel tube on the behaviour and ultimate strength; (iv) to investigate the effect of load eccentricity on the behaviour and ultimate strength; and, (v) to determine if providing longitudinal steel reinforcement to concrete filled steel columns has a significant effect on the ultimate strength of such columns. To achieve these objectives, tests were carried out on 26 columns, each with a square cross-section 150 x 150 mm and height 1000 mm, as seen in Figure 1. The dimensions of the specimens were deliberately chosen to avoid overall buckling of the specimens as this investigation is limited to short columns. Besides, the specimens were chosen with ratio of steel tube cross section area to concrete area, A_{steel}/A_{total}, above the value of 0.04 the minimum required by American Institute of Steel Construction (cited in Reference 5) to constitute a composite column. The tested columns were divided into three groups. Group (I) contains 16 concrete filled steel box columns, the details of which are shown in Table 1. Columns C-1, C-2, and C-3 are CFT columns having steel box wall thickness, t, of 2.5, 4.5, and

6.0 mm, respectively. For these columns, no steel dowels are used. Columns C-4, C-5, and C-6 are typically the same as columns C-1, C-2, and C-3 except for the presence of steel dowels (studs) ϕ12mm @ 10 cm., 30 mm in length, welded to the four inner faces of the steel tube. These had normal strength concrete with an average cube compressive strength f_{cu}= 280 kg/cm^2 (28 MPa, 4.0 ksi). Columns C-7 to C-12 are typically the same as columns C-1 to C-6 except that they contained high strength concrete with an average cube compressive strength of 620 kg/cm^2 (62 MPa, 8.8 ksi). Columns C-1 to C-12 were tested under concentric compressive loads. Columns C-13 and C-14 are typically the same as columns C-2 and C-3 except that they were tested under eccentric compressive loads (e=25 mm in single curvature). Also, columns C-15 and C-16 are typically the same as C-8 and C-9 except that they were tested under eccentric compressive loads (e=25 mm in single curvature). Group II contains 8 reinforced concrete columns, the details of which are shown in Table 2. For columns C-17 to C-20, normal strength concrete is used with an average cube compressive strength f_{cu} = 280 kg/cm^2 (28 MPa, 4.0 ksi). For column C-17, no longitudinal or transverse steel reinforcement is provided. Column C-18 had longitudinal reinforcement of 4 ϕ 10 mm in addition to stirrups of ϕ 8 mm @ 15 cm. The longitudinal reinforcement is increased for column C-19 to 4 ϕ13 mm whereas the stirrups is kept the same as in column C-18. Stirrups are increased for column C-20 to ϕ 8 mm @ 10 cm whereas the longitudinal reinforcement is kept the same as in column C-19. Columns C-21 to C-24 are typically the same as columns C-17 to C-20 except that high strength concrete is used for columns C-21 to C-24, with an average cube compressive strength f_{cu} = 620 kg/cm^2 (62 MPa, 8.8 ksi). All columns in group (II) are tested under concentric compressive loads. Group (III) contains 2 reinforced concrete filled steel columns, the details of which are shown in Table 3. Both columns are made of high strength concrete and are tested under concentric compressive loads. Column C-25 is provided with 4ϕ13 mm as longitudinal reinforcement whereas column C-26 is provided with stirrups of ϕ8 mm @ 100 mm in addition to longitudinal reinforcement.

Table 1 Details of tested concrete-filled steel columns

Column	Thickness of steel box (mm)	Presence of steel dowels	Type of filled-in concrete	Concrete compressive strength (kg/cm^2)	Type of loading
C-1	2.5	No Dowels	Normal	280	Concentric
C-2	4.5	No Dowels	Normal	280	Concentric
C-3	6.0	No Dowels	Normal	280	Concentric
C-4	2.5	ϕ12@10cm.	Normal	280	Concentric
C-5	4.5	ϕ12@10cm.	Normal	280	Concentric
C-6	6.0	ϕ12@10cm.	Normal	280	Concentric
C-7	2.5	No Dowels	High Strength	620	Concentric
C-8	4.5	No Dowels	High Strength	620	Concentric
C-9	6.0	No Dowels	High Strength	620	Concentric
C-10	2.5	ϕ12@10cm.	High Strength	620	Concentric
C-11	4.5	ϕ12@10cm.	High Strength	620	Concentric
C-12	6.0	ϕ12@10cm.	High Strength	620	Concentric
C-13	4.5	No Dowels	Normal	305	Eccentric
C-14	6.0	No Dowels	Normal	305	Eccentric
C-15	4.5	No Dowels	High Strength	620	Eccentric
C-16	6.0	No Dowels	High Strength	620	Eccentric



a: Steel Tube
b: Concrete Filled Steel /Tube
c: CFT with Longitudinal Bars
d: R.C. Encased Steel Column
e: Conventional R.C. Column
f: Tube with Welded Studs

Figure 1 Specimens details

Table 2 Details of tested reinforced concrete columns

Column	Longitudinal reinforcement	Stirrups	Type of concrete	Concrete compressive strength (kg/cm^2)	Type of loading
C-17	---	---	Normal	280	Concentric
C-18	4ϕ10	ϕ8@15cm	Normal	280	Concentric
C-19	4ϕ13	ϕ8@15cm	Normal	280	Concentric
C-20	4ϕ13	ϕ8@10cm	Normal	280	Concentric
C-21	---	---	High Strength	620	Concentric
C-22	4ϕ10	ϕ8@15cm	High Strength	620	Concentric
C-23	4ϕ13	ϕ8@15cm	High Strength	620	Concentric
C-24	4ϕ13	ϕ8@10cm	High Strength	620	Concentric

Table 3 Details of tested reinforced concrete-filled steel columns

Column	Thickness of steel box (mm)	Longitudinal Reinforcement	Stirrups	Type of filled-in concrete	Concrete compressive strength (kg/cm^2)	Type of loading
C-25	2.5	4 ϕ 13	---	High Strength	620	Concentric
C-26	2.5	4 ϕ 13	ϕ8 @ 10 cm	High Strength	620	Concentric

All the columns included in the experimental program were tested in compression using a 300 Ton (3000 kN) testing machine, shown in Figure 2. The columns can be considered with their 2 ends hinged (pinned columns). A top and bottom thin layer of gypsum was applied at the columns' ends to ensure that the steel tube and the concrete core were loaded simultaneously. In order to measure the strains and perimeter expansion, foil strain gauges, of type N11-FA-10-120-11 were installed at the columns' mid-heights on the exterior of the steel tube wall in both longitudinal and transverse directions relative to the load directions. For the columns tested, the load was applied in increments of 2.5 ton (25 kN), up to failure, and strains and failure mode were observed and recorded. Figure 3 shows one of the tested CFT columns after test.

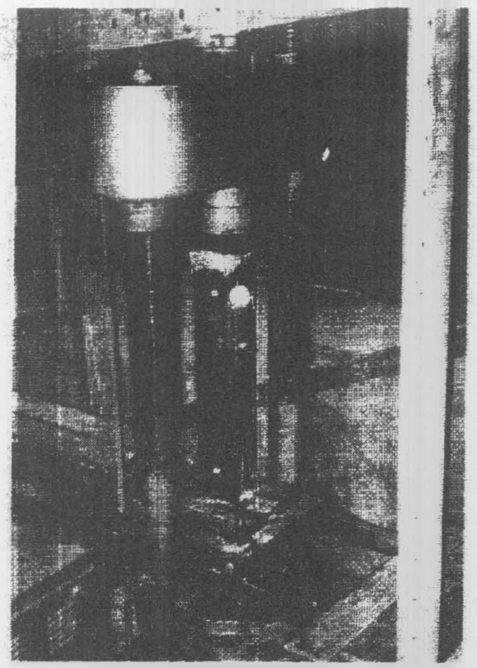


Figure 2 Test setup for CFT column

Figure 3 One of the tested CFT columns after failure

Material Properties and Concrete Mix

Two types of concrete were used throughout the experimental investigation. Twelve columns were made using normal strength concrete (NSC) whereas 14 columns were made using high strength concrete (HSC). For both types of concrete, the cement used was locally produced commercially available ordinary Portland cement, type I. Locally available natural desert sand was used as fine aggregate. For normal strength concrete, the mix proportions were 1 : 1.8 : 3.2 by weight (cement : fine aggregates : coarse aggregates). Crushed stone with maximum aggregate size of 15 mm was used as coarse aggregate, and the water cement ratio, w/c = 0.4. For high strength concrete, the water cement ratio, w/c was kept in the range 0.29, pink lime stone with maximum aggregate size of 13 mm was used as coarse aggregate, and a commercially available super-plasticizer (water reducing agent) was used to increase workability. Plain round steel bars of diameter 10 mm and 13 mm were used for longitudinal reinforcement for reinforced concrete columns whereas bars of diameter 8 mm were used for the stirrups. The steel boxes were made using mild steel with a thickness of 2.5, 4.5, and 6.0 mm The mechanical properties of the steel were determined using tensile coupon tests. Six tensile coupon tests were done and the measured average yield stress was 2730 kg/cm^2 (273 MPa). Shear studs were welded to the steel shell interior with fillet weld around the circumference of the studs. The shell (tube) halves were then welded together along the whole cut length (two longitudinal seam welds).

TEST RESULTS: COMPARISONS AND DISCUSSION

Test results including load-strain relationships; failure loads; and failure modes and comparisons between test results of various specimens are presented in Figures 3 to 9 and Table 4. Besides, Table 5 provides comparisons between test results and some codes theoretical predictions.

Comparisons of Columns' Performance
Control Specimens

Column specimens without steel shell were considered control specimens, i.e., C-17 to C-20 for NSC. For HSC column, specimens C-21 to C-24 were considered control specimens. Some of these control specimens are plain concrete columns, i.e., C-17 for NSC; and C-21 for HSC, others are reinforced concrete columns, e.g., C-18 to C-20 for NSC and C-22 to C-24 for HSC. In turn, the reinforced concrete column specimens have different longitudinal steel ratios, e.g., C-18, C-19 for NSC; and C-22 and C-23 for HSC, besides different stirrups spacing, e.g., C-19 and C-20 for NSC.

Columns with Different Shell Thickness
Concrete Filled Columns

NSC filled tube columns C-1, C-2 and C-3 with shell thickness, t= 2.5, 4.5, and 6.0 mm had failure loads of 77, 93 and 135 tons, respectively. This is an increase in load carrying capacity of 75, 111and 207%, respectively compared to the control specimen C-17 without a steel shell with failure load of 44 tons. Increasing the steel shell thickness from 2.5 mm to 4.5 and 6.0 mm for NSC filled tube columns resulted in an increase in the load carrying capacity by 20% and 75%, respectively over that of C-1 with shell thickness of 2.5 mm HSC filled tube columns C-7, C-8 and C-9 with shell thickness, t= 2.5, 4.5, and 6.0 mm, respectively, had failure loads of 135, 155 and 190 tons, respectively. This is an increase in load carrying capacity of 85, 112 and 160%, respectively compared to the control specimen C-21 with failure load of 73 tons. Increasing the steel shell thickness from 2.5 mm to 4.5 and 6.0 mm resulted in an increase in the load carrying capacity by 15% and 41%, respectively over that of C-7 with shell thickness of 2.5 mm

Hence, for the studied cases:

- -The existence of steel shell is approximately equally effective in enhancing the load carrying capacity for both NSC and HSC filled tube columns.
- -The effect of increasing the shell thickness is more pronounced for NSC filled tube columns rather than for HSC ones.

Columns with Studded and Unstudded Steel Shell

For NSC filled columns the presence of internal studs (dowels) welded to the steel shell increased the load carrying capacity by about 10% over their counterpart columns with no dowels, i.e., for the used t=2.5, 4.5 and 6.0 mm (see comparisons between C-4, C-5 and C-6 and columns C-1, C-2 and C-3, respectively). For HSC filled columns C-10, C-11 and C-12 the presence of studs (dowels) increased the load carrying capacity by about 4, 10, and 9 % over their counterpart columns with no dowels, i.e., C-7, C-8 and C-9, for the used t= 2.5, 4.5 and 6.0 mm, respectively. Columns with studded shells had slightly greater ultimate strength than columns without studs. Shells equipped with studs slightly enhanced the frictional bond between the concrete core and steel shell and decreased the tendency of slip between them and thereby increasing the composite action and the strength of the column (Figure 4). In this respect one has to recall that Shams *et al.* [5] reported that there were doubts about the effect of bond strength on the overall response of CFT. Besides, Khalil debated in Reference 16 whether significant strength was lost in CFT member once the bond between materials broke down.

Columns With Normal And High Strength Concrete Cores

Concrete Filled Columns without Studs

Increasing the strength of the used concrete from (28 MPa to 62 MPa) to fill the steel tubes resulted in increasing the load carrying capacity by 75, 66 and 41% for shell thickness, t=2.5, 4.5 and 6.0 mm, respectively (i.e. compare C-1 to C-7; C-2 to C-8; and C-3 to C-9; in Figure 5). If the columns were plain concrete with no steel tube such increase is 66% (compare C-17 to C-21). The increase in load carrying capacity reduces as the shell thickness increases.

Concrete filled columns with studs

Increasing the strength of the used concrete from (28 MPa to 62 MPa) to fill the steel studded shell tubes resulted in increasing the load carrying capacity by 65, 69 and 40% for shell thickness t=2.5, 4.5 and 6.0 mm, respectively (compare C-4 to C-10, C-5 to C-11 and C-6 to C-12). Such increase is similar to columns without studs for both NSC and HSC.

Ordinary reinforced concrete columns

Increasing the strength of the used concrete from (28 MPa to 62 MPa) for ordinary reinforced concrete columns (with no steel tube) resulted in increasing the load carrying capacity by about 54% (compare C-22 to C-18; C-23 to C-19; and C-24 to C-20) for the studied cases.

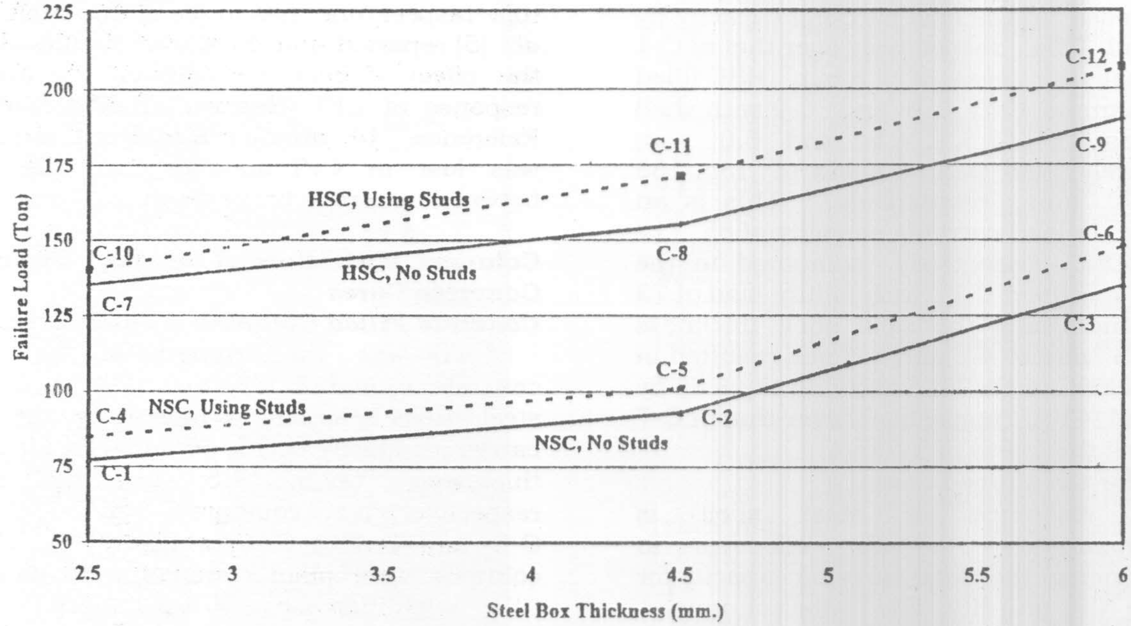


Figure 4 Effect of using studs on failure loads of CFT columns

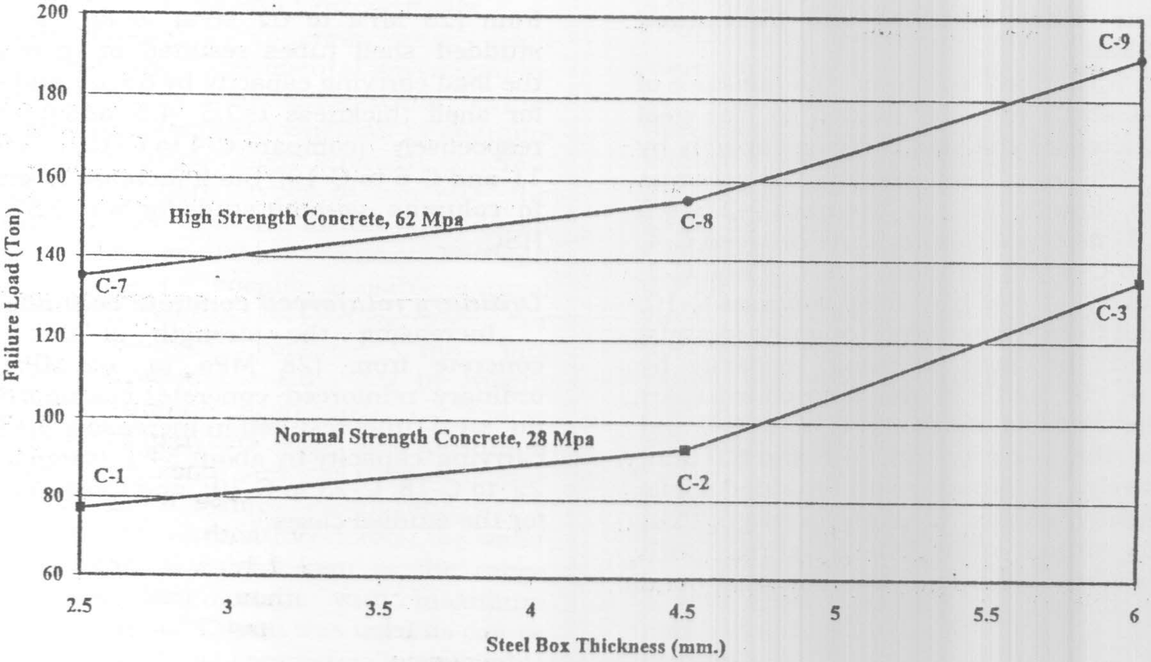


Figure 5 Failure loads of CFT columns-normal strength VS. high strength concrete

Table 4 Failure loads of tested columns

Column	Failure Loads Ton (kN)	Failure Mode
C-1	77 (770)	N.
C-2	93 (930)	S.B.
C-3	135 (1350)	S.B. + W.
C-4	85 (850)	S.B.
C-5	101 (1010)	S.B. + W.
C-6	148 (1480)	N.
C-7	135 (1350)	C.C.
C-8	155 (1550)	N.
C-9	190 (1900)	C.C. + W.
C-10	140 (1400)	S.B.
C-11	171 (1710)	S.B. + C.C.
C-12	207 (2070)	S.B. + W.
C-13	63 (630)	C.C. + W.
C-14	121 (1210)	N.
C-15	136 (1360)	S.B.
C-16	137 (1370)	W.
C-17	44 (440)	C.C.
C-18	61 (610)	C.C. + L.B.
C-19	65 (650)	C.C.
C-20	66 (660)	C.C.
C-21	73 (730)	C.C.
C-22	90 (900)	C.C. + L.B.
C-23	100 (1000)	C.C. + L.B.
C-24	106 (1060)	C.C. + L.B.
C-25	145 (1450)	S.B. + C.C.
C-26	145 (1450)	S.B. + W.

S.B. = Steel Shell Local Buckling
C.C. = Concrete Crushing
W. = Weld Failure (opening)
N. = No External Features
L.B. = Longitudinal Bars Buckling

Comparisons Between R.C. Encased Steel Columns And CFT Columns

Effect of longitudinal steel

The presence of longitudinal steel reinforcement in HSC filled tube columns compared to plain concrete filled tube columns increased the load carrying capacity by about 7.5 % for shell thickness of t= 2.5 mm (e.g. compare C-25 to C-7). Hence the presence of the used longitudinal steel in CFT is not that pronounced in increasing the load carrying capacity. Besides, the inclusion of transverse steel stirrups in addition to longitudinal bars in HSC reinforced steel encased columns did not influence the load carrying capacity, and the effect of changing stirrups is not pronounced (compare C-25 to C-26). However, such observations were based on limited specimens and may need further verification.

Ordinary Reinforced Columns And CFT Columns

Normal strength CFT columns with steel shell thickness t=2.5, 4.5 and 6.0 mm have more load carrying capacity than ordinary normal strength reinforced concrete columns with the used longitudinal and transverse (stirrups) reinforcement, e.g., compare C-1 to C-20. This applies also to the HSC case, e.g., compare C-7 to C-24. Hence apparently CFT have greater axial load carrying capacity than ordinary reinforced concrete columns, and hence are more effective and stronger than ordinary R.C. columns. However, one should note that the used shell thickness, i.e., for t=2.5 mm, has an equivalent area approximately 3 times that of the used longitudinal bars (steel shell equivalent area=15.0 cm^2, while the used longitudinal bars area=5.0 cm^2 approximately). According to many codes (e.g. ACI–318-95 [20]) the contribution of the used steel whether as a tube (shell) or longitudinal bars depends on their respective areas. Comparing C-23 the ordinary reinforced HSC column with its counterpart column C-26 with a steel shell of t=2.5 mm, the existence of the steel shell increased the load carrying capacity by 45%.

Eccentric versus Concentric Loading For CFT Columns

For the used eccentricity (e=25 mm about a single principal axis) CFT columns with NSC showed a decrease in the load carrying capacity relative to their concentrically loaded counterparts. Such decrease in capacity is 32 and 10% for t=4.5 and 6.0 mm, respectively (compare C-13 to C-2; and C-14 to C-3). For the case of HSC such reduction is 12 and 28% for t=4.5 and 6.0 mm, respectively (compare C-15 to C-8; and C-16 to C-9). Hence eccentrically loaded CFT has less load carrying capacity than concentrically loaded ones, as naturally expected. It was strange to note that the failure loads for both C-15 and C-16 (eccentrically loaded high strength CFT are almost identical although they have different shell thickness t=4.5 and 6.0 mm, respectively. However one must note that C-15 failed by local buckling of the steel shell while for C-16 the weld failed prematurely and the test was stopped. One must note that the concentrically loaded HSC C-8 and C-9 counterparts for C-15 and C-16 do not show such trend. Besides, eccentrically loaded NS CFT C-13 and C-14 show considerable difference in their load carrying capacity, due to the difference in shell thickness.

Strain Measurements

Steel shell longitudinal and transverse strain readings at the shell exterior mid-height versus load level are given for some specimens (Figures 6, 7 and 8). The figures depict the effect of various parameters, i.e., steel shell thickness; existence of studs; and use of HSC, on the strain readings. Besides, the absolute ratio between transverse (perimeter) to longitudinal steel shell, i.e. strain ratio, versus the axial compression ratio (defined as load level divided by failure load) is also provided in Figure 9.

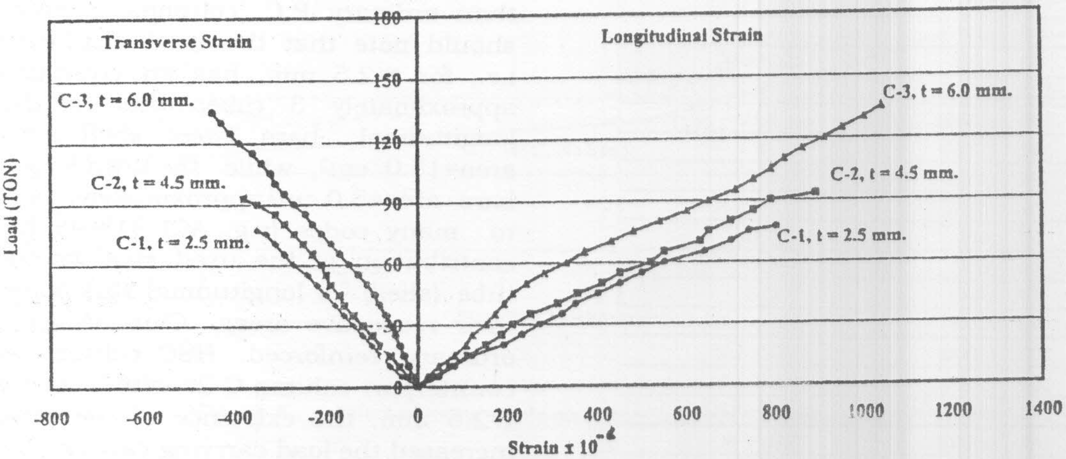


Figure 6 Effect of steel box thickness on steel strain

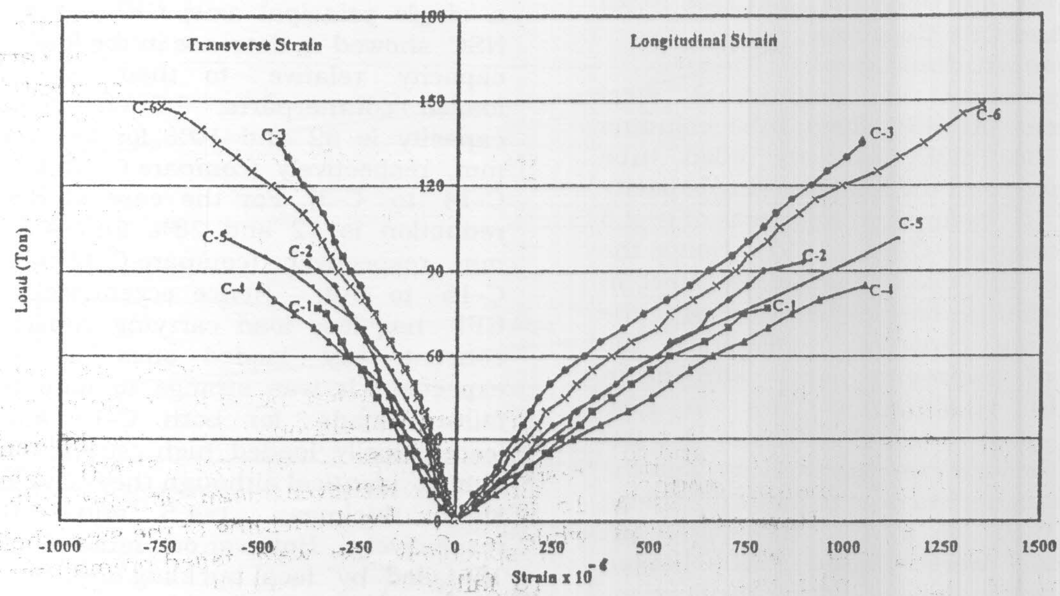


Figure 7 Effect of using studs on steel strain

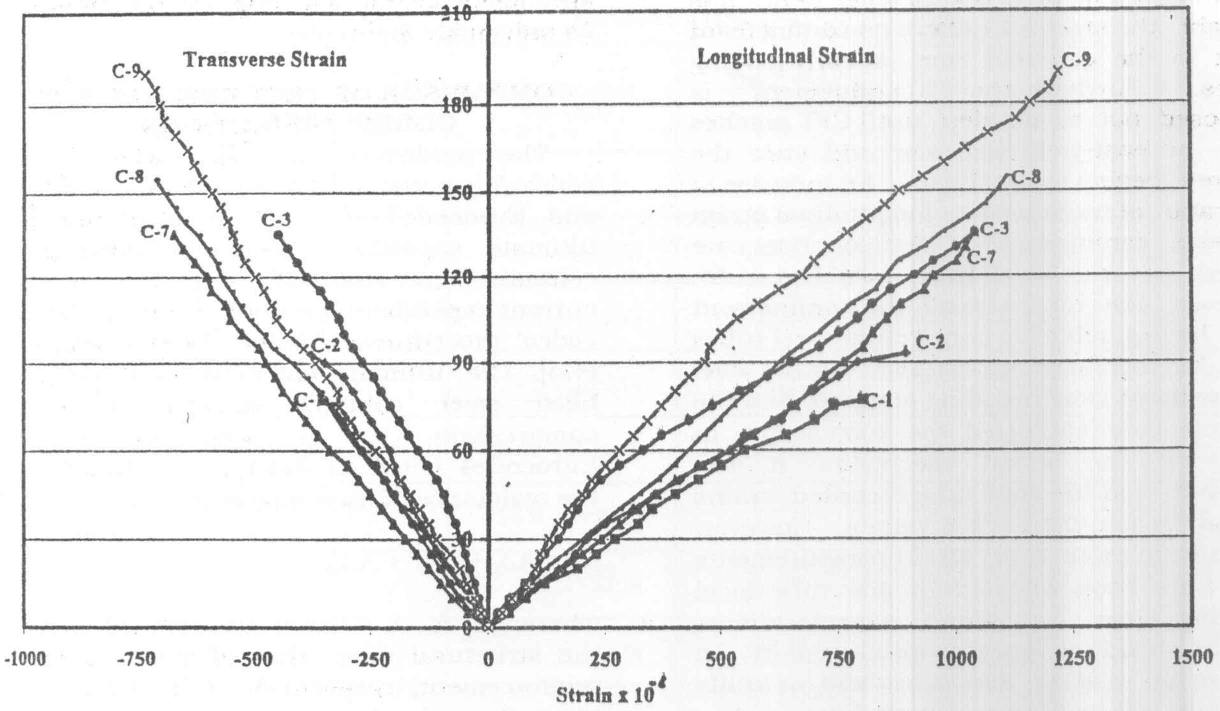


Figure 8 Effect of using high strength concrete on steel strain

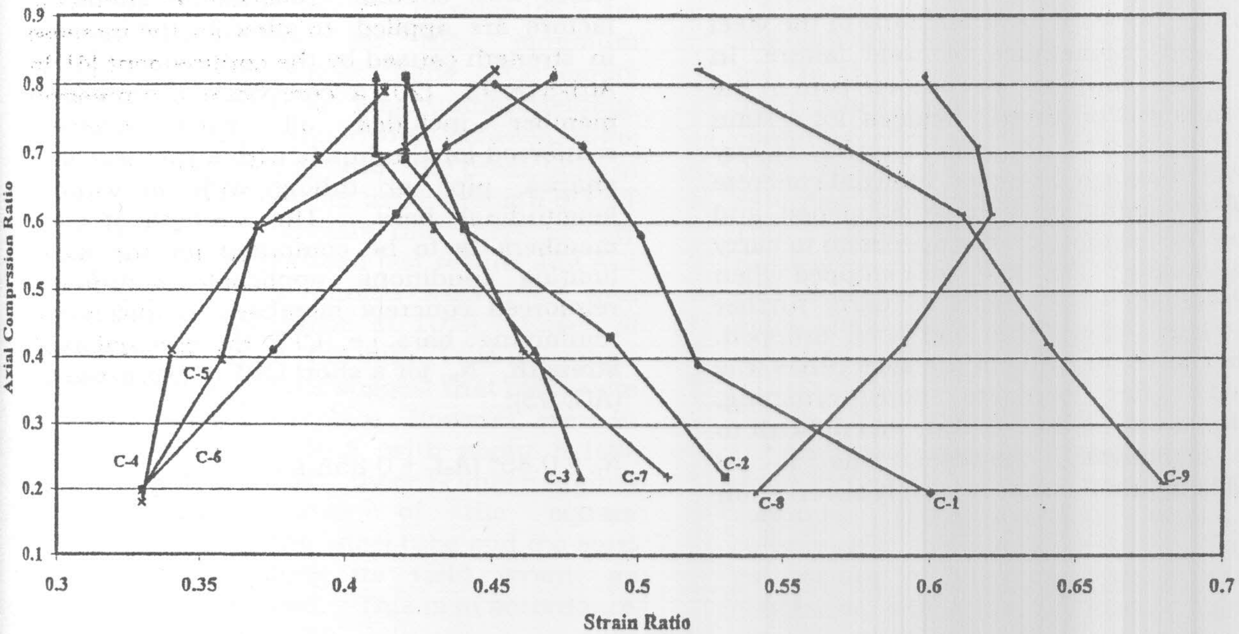


Figure 9 Axial to failure load ratio with respect to strain ratio

As previously noted, Poisson's ratio for concrete is less than that of steel, hence it is thought the steel tube offers no confinement effect to the concrete core at early loading stages. Furthermore, confinement is supposed not to develop until CFT reaches yield or post-yield behavior and once the concrete begins to crush [16]. An increase in the ratio of transverse to longitudinal strain suggests confinement of the concrete core offered by the steel tube, (see Figure 9). However, one has to recall that confinement may be negligible for noncircular steel tubes [3]. An increased strain level for the steel tube without local buckling suggests that the concrete core stabilized the tube wall. In most of the tested specimens, it was observed that the steel tube recorded strains did not reach their yield values. However, one has to note that strain measurements may have been affected by the tube local buckling near the strain gage locations. Besides, residual stresses produced in the process of welding the tubes and/or studs may have affected the strain readings.

Failure Modes

Failure modes of the tested short specimens are reported in Table 4 and can be generally characterized by concrete crushing; outward local buckling of the steel shell; and sometimes by weld failure. In some cases slippage of concrete core in the steel tube was observed. Besides, for certain cases failure was not accompanied by any external damage, however, internal concrete failure (crushing) may have happened and led to the inability on the specimen to carry further loads. The test was stopped when the specimen was unable to carry further loads, and when the load level dropped. Whether local buckling of the steel tubes was induced after concrete core crushing, simultaneously or before that was difficult to detect accurately, however this is in accordance with other research observation [5]. Because of the chosen dimensions of the specimens overall buckling was not induced, as previously mentioned.

COMPARISON OF TEST RESULTS WITH CODES' PREDICTIONS

The predictions of some widely used codes for design of CFT; namely ACI 318 [20] and Eurocode4 (EC4) [19] for calculating the ultimate capacity of concrete filled steel columns were assessed using some of the current experimental results (Table 5). Other codes' procedures may be found elsewhere [4,5]. The ultimate load capacity of concrete filled steel columns subjected to axial compression, N_u, is presented in the Eurocode4 (cited in Reference 4) by adding the resistance of its components thus:

$$N_u = A_s f_y + A_c f_c' + A_r f_{sk} \qquad (1)$$

where; A_s, A_c, A_r = the cross-sectional area of the structural steel, the concrete, and the reinforcement, respectively; f_y, f_{sk} = the yield strength of the structural steel and reinforcement respectively; and, f_c' = the characteristic cylinder strength of concrete (taken as 0.85 f_{cu}). In the previous equation the partial safety factors were omitted. For tubes with circular cross-section additional factors are applied to allow for the increase in strength caused by the confinement [4]. In ACI-318-95 [20] a composite compression member includes all such members reinforced longitudinally with structural steel shapes, pipe or tubing with or without longitudinal bars. The strength of such members is to be computed for the same limiting conditions applicable to ordinary reinforced concrete members. In absence of reinforcing bars, i.e., CFT, the nominal axial strength, N_n, for a short CFT is expressed as (ACI-95):

$$N_n = 0.85^* (A_s f_y + 0.85 A_c f_c') \qquad (2)$$

Table 5 Comparison of experimental failure loads for tested columns to those predicted by codes of practice

Column	Experimental Failure Loads Ton (kN)	Eurocode 4 [19]		ACI 318-95 [20]	
		Failure Loads Ton (kN)	$\frac{V_{exp}-V_{code}}{V_{exp}}$ (%)	Failure Loads Ton (kN)	$\frac{V_{exp}-V_{code}}{V_{exp}}$ (%)
C-1	77.0 (770.0)	94.5 (945.0)	-22.7	73.5 (735.0)	4.5
C-2	93.0 (930.0)	127.3 (1273.0)	-36.9	101.3 (1013.0)	-8.9
C-3	135.0 (1350.0)	151.8 (1518.0)	-12.4	122.2 (1222.0)	9.5
C-7	135.0 (1350.0)	159.5 (1595.0)	-18.1	120.5 (1205.0)	10.7
C-8	155.0 (1550.0)	192.3 (1923.0)	-24.1	148.3 (1483.0)	4.3
C-9	190.0 (1900.0)	216.9 (2169.0)	-14.2	170.0 (1700.0)	10.5
C-25	145.0 (1450.0)	172.2 (1722.0)	-18.8	131.3 (1313.0)	9.4
C-26	145.0 (1450.0)	172.2 (1722.0)	-18.8	131.3 (1313.0)	9.4

All fibers are assumed to be subject to 0.003 strain. ACI-318 [20], as many other approaches [5] does not account for possible enhancement in strength or ductility due to confined concrete in CFT. Besides there are some restrictions for tube minimum thickness to prevent buckling of an empty steel shell prior to longitudinal yielding. Table 5 shows comparisons between some experimental results and the predictions of the previous codes for axial strength of CFT columns. It is obvious that there is very good agreement between test results and ACI predictions for the studied cases of NSC and HSC FT (in the range of 10%). Cases of higher loads than the sum of the individual constituent materials suggest that composite action has taken place. However even in such cases in Table 5 (with about 5-10% increase in capacity) there is only marginal confinement because of the square configuration of the steel tube and the steel shell not reaching its yield strain, as previously discussed. This is in accordance with other findings [6].

On the other hand, the Eurocode4 [19] greatly overpredicts the axial strength for the considered cases. This might be partially attributed to the fact that in Eurocode4 the 0.85 factor applied to f'_c to relate the strength obtained from a standard cylinder test to the uniaxial strength is omitted for filled tubes probably because of a suggested confining effect of the tube [4]. Hence, the 0.85 factor (i.e., 0.85 * f'_c) seems to be essential for determining CFT axial strength. This might call upon the need to modify Eurocode4 predictions for axial strength of CFT especially with square shapes. However, more tests might be needed to support such observation.

CONCLUSIONS

An experimental investigation on 26 short concrete filled square steel box columns (150x150x1000 mm) under compressive loads was presented. The main parameters included are: testing under both concentric and eccentric loads; using both normal, NSC (28 MPa) and high strength concrete HSC (620 kg/cm^2, 62 MPa, 8.9

ksi.); steel tubes thickness (t= 2.5, 4.5 and 6.0 mm); inclusion of longitudinal steel bars; and existence of shear studs (dowels). The influence of the previous parameters on the axial strength and structural response were investigated. Besides, the longitudinal and transverse strains were monitored and the failure modes reported. Comparisons between columns with different steel tube thicknesses, concrete strengths, and shear studs were made. The behaviour of concrete filled tubes CFT were studied and compared to their counterpart conventional reinforced concrete columns. Besides, comparisons with available codes' predictions for the axial strength of CFT columns [19-20] were performed. In view of the studied cases and variables the following conclusions can be drawn:

1. Increasing the steel shell thickness increases the axial strength of CFT columns, as naturally expected. The effect of increasing the shell thickness is more pronounced for NSC filled tube columns than for HSC ones.
2. The effect of using internal longitudinal reinforcement in CFT columns slightly increased the axial capacity of the tested specimens. The increase is proportional to the cross section of the used steel reinforcement, whether shell (tube) or longitudinal bars.
3. For the tested square steel shells, internal studs (dowels) only slightly increased the axial load capacity of CFT columns (in the range of 10%) since the bond between concrete core and steel tube was slightly enhanced.
4. Steel tubes with square shapes only slightly increased concrete confinement and hence the axial load carrying capacity of CFT columns was marginally increased over that of the columns' independent constituent materials. Hence the effect of confinement is not that pronounced in square tubes. This can be attributed to the square (noncircular) configuration of the steel tube, besides, the steel shell not reaching its yield strain. This in accordance with other research findings that confinement may be negligible for noncircular steel tubes. Nevertheless, although the concrete compressive strength will not be augmented by confinement, however the steel tube helps preventing the brittle failure that is normally associated with unconfined high strength concrete.
5. CFT columns with HSC resulted in a larger axial capacity than when using NSC as naturally expected.
6. Eccentrically loaded CFT columns have less load carrying capacity than concentrically loaded ones, as expected.
7. Comparisons between experimental test results with ACI 318-95 [20] axial strength predictions for CFT columns are in close agreement. Hence, ACI predictions for axial strength seem adequate for square CFT columns. However, Eurocode4 [19] overpredicts the axial strength of square CFT columns partially because it omits the 0.85 factor for concrete compressive strength, calling upon the need to revise its predictions, at least for noncircular CFT columns.
8. It seems adequate to calculate the axial strength of square CFT columns based on adding the resistances of its components (concrete core; and steel, i.e. steel tube and longitudinal reinforcing steel bars if any), without augmenting the strength due to confinement effects. This is in accordance with ACI 318-95 [20].

REFERENCES

1. J. Hooper, C. Roeder, R. Klemencic and K. Nordquist, "Best of Both Worlds", ASCE Civil Engineering Magazine, Vol. 1, January, pp. 40-43, (1999).
2. A. Kilpatrick and V. Rangan, "Tests on High Strength Concrete Filled Steel Tubular Columns", ACI Structural Journal, Vol. 96, No. 2, pp. 268-274, (1999).
3. W. Zhang and B. Shahrooz, "Strength of Short and Long Concrete Filled Tubular Columns", ACI Structural Journal, Vol. 96, No. 2, pp. 230-238, (1999).
4. Association for International Cooperation and Research in Steel-Concrete Composite Structures. "Concrete Filled Steel Tubes: A Comparison of

International Codes and Practices", ASCCS Seminar, Innsbruck, pp. 134, September (1997).
5. M. Shams and M.A. Saadeghvaziri, "State of the Art of Concrete Filled Steel Tubular Columns", ACI Structural Journal, Vol. 94, No. 5, pp.558-571, (1997).
6. S. Schneider, "Axially Loaded Concrete-Filled Steel Tubes", ASCE Structural Journal, Vol. 124, No. 10, pp.1125-1138, (1998).
7. J. Gardner and R. Jacobson, "Structural Behaviour of Concrete Filled Steel Tubes", ACI Journal, No. 64-38, pp. 404-413, (1967).
8. R.W. Furlong, "Design of Steel-Encased Concrete Beam Columns", ASCE, Journal of Structural Division, Vol. 94, No. ST1, pp. 267-281, (1968).
9. P.K. Neogi and H.K. San, "Concrete Filled Tubular Steel Columns Under Eccentric Loading", ASCE Journal of Structural Engineering, Vol. 47, No. 5, (1969).
10. R.B. Knowles and R. Park, "Axial Load Design For Concrete Filled Steel Tubes", Proceedings of the American Society of Civil Engineers, Vol. 96, No. ST10, pp. 2125-2153, (1970).
11. H. Ge and T. Usami, "Strength of Concrete Filled Thin Walled Steel Box Columns: Experiment", ASCE Journal of Structural Engineering, Vol. 118, No. 11, pp.3036-3054, (1992).
12. R. Bridge and J. Webb, "Thin Walled Circular Concrete Filled Steel Tubular Columns", ASCE Composite Construction in Steel and Concrete II, June 14-19, pp. 634-649, U.S.A. (1992).
13. V. Rangan M. Joyce, "Strength of Eccentrically Loaded Slender Steel Tubular Columns Filled With High Strength Concrete", ACI Structural Journal, Vol. 89, No. 6, pp. 676-681, (1992).
14. H.A. Zien El-Din, "Stability and Post-Buckling Behaviour of Concrete Filled Steel Square Tubular Columns", Ph.D. Thesis, Department of Architecture, Faculty of Engineering, Kyushu University, Japan, pp.128, (1993).
15. H.A. Zien El-Din, C. Matsui and K. Tsuda, "Stability and Post-Buckling Behaviour of Concrete Filled Square Tubular Columns", Journal of Structural Engineering, Vol. 39B, pp. 323-334, (1993).
16. J.F. Hajjar and B.C. Gourley, "Representation of Concrete Filled Steel Tube Cross-Section Strength", ASCE Journal of Structural Engineering, Vol. 122, No. 11, pp.1327-1336, (1996).
17. M.A. Bradford, "Design Strength of Slender Concrete Filled Rectangular Steel Tubes", ACI Structural Journal, Vol. 93, No. 2, pp. 229-235, (1996).
18. P. Boyd, W. Cofer and D. Mclean, "Seismic performance of Steel-Encased Concrete Columns Under Flexural Loading", ACI Structural Journal, Vol. 92, No. 3, pp. 355-364, (1995).
19. Eurocode4, "Design of Steel and Concrete Structures, Part 1.1, General Rules and Rules for Buildings" DD ENV 1994-1-1: 1996, British Standards Institution, London, (1996).
20. "Building Code Requirements of Reinforced Concrete", American Concrete Institute (ACI 318-1995), USA (1996).

Received August 18, 1999
Accepted November 28, 1999