© IJEDR 2018 | Volume 6, Issue 2 | ISSN: 2321-9939

Behavior of the Concrete Core at the Critical Zones of Concrete Filled Steel Tube Columns after Using CFRP Composites as Additional Reinforcement

Ibrahim Abd-Elaal M. Abd-Elwahed[1], Mohamed F.M.Fahmy[2], Zainab.I.Abdelshafy[3],

Abd-El Rahman M. Ahmed[4], SedkyA.Tohamy[5]

[1]Demonstrator, [2]Assistant Professor of Structural Engineering, [3]Doctor of Structural Engineering,
[4]Professor of Structural Engineering, [5]Professor of Structural Engineering
[1]Civil Engineering Department, Faculaty of Engineering Assiut University, Assiut, Egypt

***Abstract*—This study proposed Carbon fiber reinforced polymers (CFRP) as additional transverse reinforcement at the critical zones of concrete filled steel tubular (CFST). An experimental study consisted of five main sets of specimens representing the ends of columns, such as those merging in through beam-column connections, was conducted. Each main set of specimens investigated the behavior of the concrete core for a specific case of CFST or CFRP wrapped CFST (CFCFST), and each main set comprised three similar specimens to get more accurate results. All specimens were 160 mm external diameter and 320 mm height and had the same concrete grade. The thicknesses of the steel tubes used were 2 and 3mm. The numbers of (CFRP) layers used were one and two layers. The results showed that one and two CFRP outer layers added to CFST greatly improved the concrete compression. Response showed 29% and 54% increase in the concrete core compressive strength, respectively. The increase in the steel tube thickness from 2mm to 3 mm caused 20% increase in the concrete core compressive strength. A new analytical model with a sufficient accuracy was driven to predict the concrete core strength for both CFST and CFCFST cases.**

***Index Terms*—Concrete, Steel tube, Fiber reinforced polymer (FRP), Confinement, Buckling.**

I. INTRODUCTION

Concrete-filled steel tube columns CFST become one of the essential types of columns due to its versatile advantages [1, 3].The connection type between CFST columns and the beams affect the behavior of the CFST. As shown in **Figure (1.a)** when the continuous beam passes through continuous CFST column (through-beam through-column connection) the load is directly acting vertically on the steel tube and concrete core together, so the steel tube performs as the longitudinal and transverse reinforcement. The steel tube of CFST fails via local buckling just below connections due to the combination of vertical compression and lateral tension stresses or may fail by local buckling at the mid-height of column [4, 6]. For this type of columns, the column cross section must be wider than the beam, so the diameter of the steel tube is difficult to be changed for higher floors when the load on column became lower. To improve the confinement of concrete for CFST, so the steel tube stops just below and above the beams to resist the lateral dilation of concrete without axial stresses, the axial stresses can be resisted by vertical reinforcement bars: through beam-column connection, in which the beam transfers the load only to the concrete core as shown in **Figure (1.b)**. Vertical stresses transfer to column concrete core by vertical steel bars and bearing on concrete, and the steel tube stands as the transverse reinforcement. The main aim of using the steel tube is to make the confinement of concrete and to enhance the concrete core strength. Based on experimental investigations by several researchers [7, 8], the through beam-column connection showed a better performance than that of the through-beam through-column connection. The same type of connection has been proposed for bridge columns at the column-footing connection [9].

Columns under seismic action may fail via crushing of concrete at the connection between footing and column or beam and column due to compression stress concentration. To enhance the performance of the CFST columns in the through-beam through-column connection, the thickness of the whole continuous column should be increased which in turn increases the own weight of the column due to steel tube constant thickness. However, for the through beam-column, the steel tube thickness and column cross section can be easily changed for each floor [10].

Alternatively, fiber reinforced polymers (FRPs) have been introduced for application in civil engineering due to its elastic performance, high strength to weight ratio, and distinctive properties in resisting the effects of harsh environmental conditions. Carbon FRPs (CFRPs) have been adopted by several researchers as outer jackets to enhance the strength and to delay the buckling of the steel tubes of CFST columns in the first type of connection (through-beam through-column connection) [11, 13]. Others studied the effect of FRP outer tube to enhance the performance of such CFST columns [14, 17]. They showed that adding CFRP at the place of stress concentration, as shown in **Figure (1.c)**, make an upgrade to the concrete core strength.

The present research concerned with studying the behavior of concrete core of these types of CFST columns. Fifteen specimens were fabricated and tested to failure under axial loading. The specimens were divided into five main sets every set consists of the same three specimens to get their average result.

The first set represents (through-beam through-column connection) continuous beam passes through the continuous column so that three specimens were fabricated and tested by loading the concrete and steel together. The second case represents (through-beam connection) so that three specimens were fabricated and tested to failure by loading the concrete core only. The third set represents an extension of the second set, where the concrete core was loaded but with a thicker steel tube to study the effect of steel tube thickness on such columns. The fourth set of specimens represents CFST with CFRP jackets added to the positions of stress concentration when the concrete core is only loaded. The fifth set is the same as case four but represents more of CFRP layers added to study its effect on the behavior of the concrete core, as shown in **Figure (1)**.

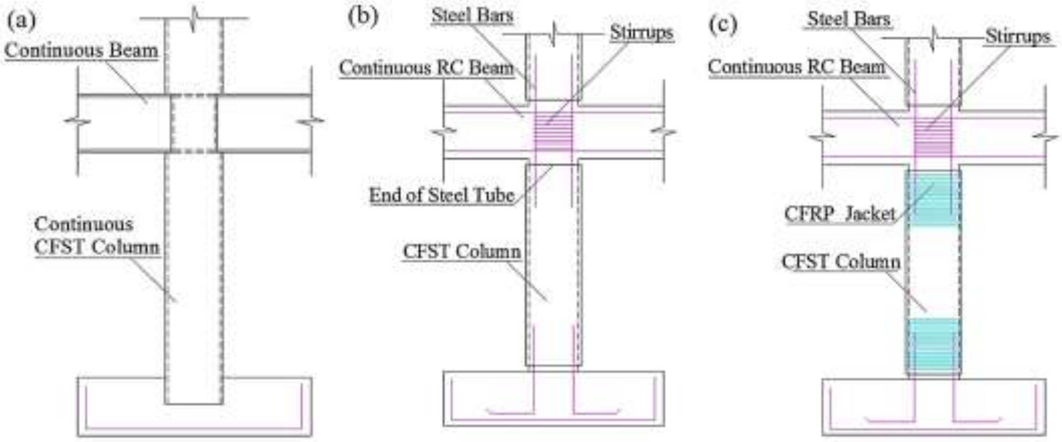

Figure (1): (a) Continuous beam passes through continuous CFST column. (b) CFST column connected with continuous reinforced concrete beam. (c) CFRP jackets added to CFST columns.

II. EXPERIMENTAL PROGRAM

Test specimens

Fifteen specimens were fabricated and tested to failure. The fifteen specimens were divided to five main sets of specimens, each set consist of three specimens had the same dimensions, steel tube thickness, loading type and the same number of CFRP layers to get its average results. All the specimens steel tubes were 160 mm outer diameter and 320 mm in height. All specimens filled with the same concrete of 35N/mm2 designed cubic compression strength. Set (1) consisted of three CFST specimens with steel tube thickness equal to 2mm, and the load directed to the steel tube and concrete together. Set (2) comprised three specimens CFST with steel tube thickness equal to 2mm, but the load was only introduced to the concrete core. Set (3) consisted of three CFST specimens with tubes thicknesses equal to 3mm. Set (4) included three CFRP wrapped CFST (CFCFST) specimens, in which the steel tube thicknesses was 2mm and it was wrapped with one layer of unidirectional CFRP jacket at the hoop direction. Set (5) consisted of three CFCFST specimens that were with steel tube thicknesses equal to 2mm and externally wrapped with two layers of unidirectional CFRP jacket at the hoop direction. All the details of the specimens are given in **Table 1**. The specimen designation is a letter t, which refers to the steel tube, and it is followed by a number refers to its thickness, and then the letter f represents the CFRP and it is followed by a number represents the number of CFRP layers. At last, a capital letter A, B, or C is used, which indicates the number of the specimen at the same set of specimens. Finally, the specimens subjected to an axial load applied to both the steel and concrete is marked by a star at the end of its label.

Table 1. Details of test specimens

Set	**Specimen**	**Steel tube**					**Concrete cylindrical strength (N/mm2)**	**Number of CFRP layers**
		Outer Diameter D (mm)	***Tube thickness ts (mm)***	D/t_s	f_y *(KN/mm2)*	f_u *(KN/mm2)*		
Set1	t2f0A*	160	2	80	245	400	30	0
	t2f0B*							
	t2f0C*							
Set2	t2f0A	160	2	80	245	400	30	0
	t2f0B							
	t2f0C							
Set3	t3f0A	160	3	53.33	252	411	30	0
	t3f0B							
	t3f0C							
Set4	t2f1A	160	2	80	245	400	30	1
	t2f1B							
	t2f1C							
Set5	t2f2A	160	2	80	245	400	30	2
	t2f2B							
	t2f2C							

Specimen's preparation

The steel tubes of the specimens which had the same thickness were cut from the same batch steel tube. All tubes of the specimens were cut at the required length using electrical saw then the top and bottom surfaces were smoothed and adjusted using a lathe. The steel tubes were fixed vertically on a horizontal surface. Then the concrete was mixed and cast inside the steel tubes. An electrical vibrator was used to prevent air the formation of air voids in the concrete core as shown in **Figure (2)**. For the specimens in set 4 and set 5 which wrapped with CFRP outer jackets the steel tube surface was prepared by removing dust and rust then the surface was roughened. The CFRP sheets were wrapped in the hoop direction using the wet layup process. Overlapping in the direction of the fibers was fitted with a length equal to 200mm. A ripped roller was used to ensure no air voids formations occurred during wet layup processes as shown in **Figure (3)**.

Figure 2. concrete casting process

Figure 3. CFRP wrapping using wet layup process

Material properties

Concrete mix

The Concrete mix designed to produce 28 days concrete cubic strength of 35 MPa. Quality control was made by preparing six concrete cubes with side length equal to 150 mm cast from the same concrete mix of the specimens to check the concrete cubic compressive strength. Then the cubes were tested and gave average cubic compressive strength equal to 36.9 MPa. To calculate the cylindrical compressive strength for concrete but only concrete cubes are available Eq.1 can be used to convert the cubic compressive strength of concrete to cylindrical compressive strength [10] and [18].

$$f_{co} = (.76 + .2\log_{10}\frac{f_{cu}}{19.6})\, f_{cu} \qquad \text{Eq.1}$$

In which f_{co} refers to concrete cylindrical strength and f_{cu} refers to concrete cubic strength. By using the previous equation the mean cylindrical compressive strength of concrete equal to 30 MPa.

Steel tubes

The tension test was carried out to determine the mechanical properties of steel tubes used. Three tension coupon tests were made and tested for each steel tube thickness the results were determined by taking the average results. The results of the tension coupon test are listed in the **Table (2)**.

Table 2.Experimental results of coupons tension test.

steel sheet thickness (mm)	Yield strength (MPa)	Ultimate strength (MPa)	Total Elongation %
2.0	245	400	23.4%

3.0	252	411	24.2%

CFRP Sheets

Unidirectional carbon Fiber reinforced polymer sheet (Sikawrap Hex-230C) was used. It is bonded using structural epoxy resins (sikadur-330). Mechanical properties of the CFRP are given in **Table (3)**.

Table 3. CFRP Mechanical properties

Weight gm/m2	Considering thickness Mm	Tensile strength of fibers (MPa)	Modulus of elasticity (GPa)	The total strain %
230	0.131	4500	210	1.50

Epoxy adhesive (Sikadur-330)

CFRP sheet were bonded with the steel tube using adhesive resin (Sikadur-330). The adhesive properties are listed in **Table (4)**.

Table 4. Sikadure-330 properties:

Modulus of Elasticity (MPa)	Tensile strength (MPa)	Tensile adhesive Strength
4000	30	>4 (concrete failure)

Instrumentation and testing:

All specimens were tested using a 5000 KN capacity compression testing machine. The average loading rate was 250 N/sec until the maximum load capacity. After that, the loading rate has been decreased to be 150 N/sec until the end of the test. Two linear variable displacement transducers (LVDTs) were used to measure the total vertical displacement and to check the results of the strain gauges. Two electrical strain gauges were placed on the hoop direction at the mid-height of the specimens spaced quarter the perimeter. Two electrical strain gauges were placed on the longitudinal direction of the specimen at the mid-height spaced quarter of the perimeter. For specimens of the set (1) two rigid square steel plates covered the full sections of the specimens to ensure that the steel tube and concrete are loaded together as shown in **Figure (4)**. For specimens of sets (2, 3, 4, 5), two circular rigid steel plates were placed on the upper and lower surface of the concrete. The circular plates diameters equal to the concrete core this to ensure that the load transfers only to the concrete core. The steel tube was not loaded by the testing machine during the test. **Figure (5)** shows test set up, the arrangement of strain gauges, positions of the LVDT's and the loading mechanism for sets (2, 3). **Figure (6)** shows the arrangement of the strain gauges and LVDT's for CFCFST specimens of sets (4, 5).

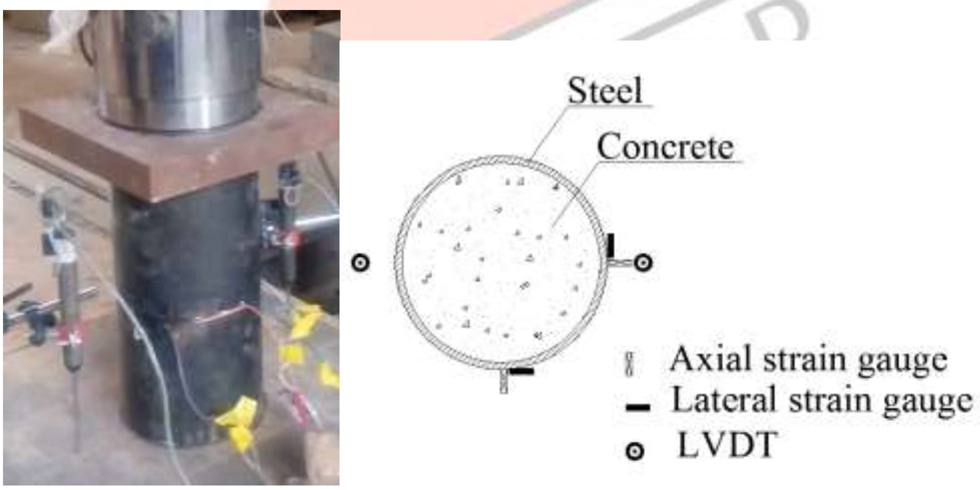

Figure 4. Test setup and arrangement of strain gauge and LVDTs for set (1) specimens'.

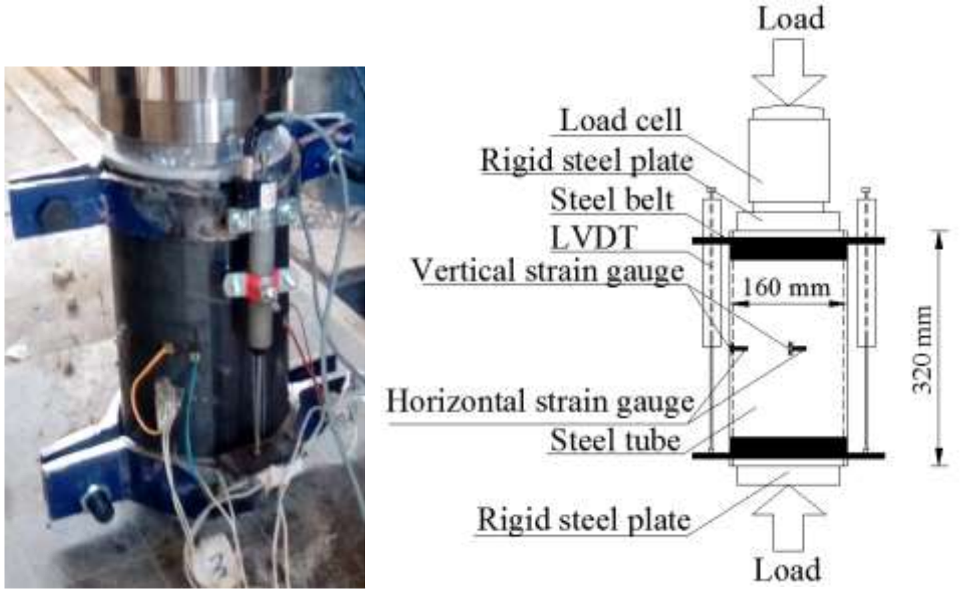

Figure 5. Test setup and arrangement of strain gauge and LVDTs for sets (2, 3) specimens'.

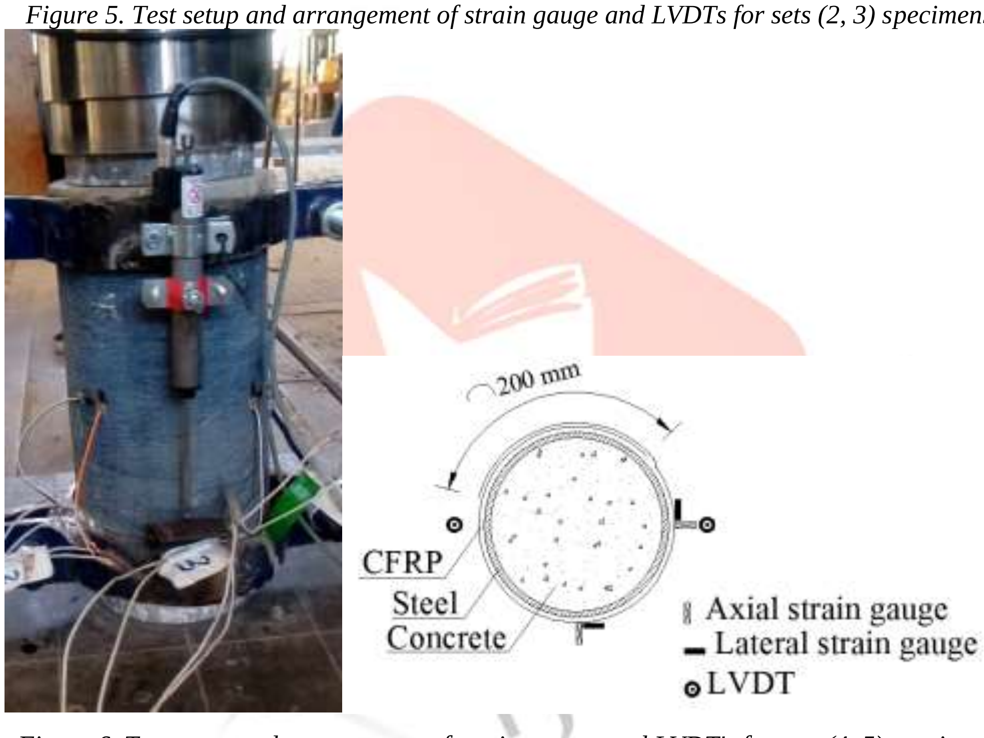

Figure 6. Test setup and arrangement of strain gauges and LVDT's for sets (4, 5) specimens'.

III. Test results and discussion

Failure mode observations

All specimens of the set (1) were with steel tube thickness equal to 2 mm. Both concrete and steel failed via outward local buckling near the top and the bottom loading square steel plates. A shear failure occurred after the maximum load capacity, then the load began to decrease, and the failure mode became more evident as shown in **Figure (7-a)**. All the concrete filled-steel tube CFST specimens of the set (2) were with steel tube thickness of 2mm. The load introduced to the concrete only where shear failure took place on an inclined plan about to 45° due to compression. One of these specimens (t2f0B) its outer steel tube was cracked because of the ring tension forces acting on the steel tube to confine the concrete core as shown in **Figure (7-b)**. After reaching the maximum load, the shear failure occurred. Then the load decreased gradually, and the failure mode became noticeable. For set (3) specimens in which steel tube thickness equal to 3mm t3f0B, and t3f0C failed via local buckling, but the third specimen t3f0A showed a shear failure on an inclined plan about to 40o from the horizontal plane as shown in **Figure (7-c)**. Set (4) consists of three CFCFST specimens, each wrapped with one layer of CFRP. When the load reached its ultimate value, a sudden rupture of the CFRP layer at the mid-height of the specimens became audible and visible simultaneously with decreasing the load suddenly. Then the behavior of the CFCFST specimens mimicked CFST third residual stage, but the stress-strain relation showed lower stresses than CFST which indicates that the concrete loses a part of its residual resistance. Set (5) consists of three CFCFST specimens, each wrapped with two layers of CFRP. When the load reaches its ultimate value, a sudden rupture of the CFRP layers at the mid-height of the specimens became audible and visible with a sudden load drop at the same time. Then the behavior of the CFCFST specimens became as CFST third residual stage. However, the stress-strain relation shows lower stresses than CFCFST wrapped with one layer of CFRP and CFST. This means that the concrete loses a part of its residual resistance with the increase of CFRP number of layers. For the set (4, 5) the rupture of the fiber jackets was caused by the tension force -created by the lateral expansion of the concrete core- which resisted by the jacket. It was evident that the outward local buckling or shear failure for CFCFST was suppressed and delayed by CFRP jacket. CFCFST showed high stability in lateral expansion more than CFST specimens. **Figure (7-d)** shows the failure mode of the set (4) specimens'. **Figure (7-e)** shows the failure mode of the set (5) specimens'.

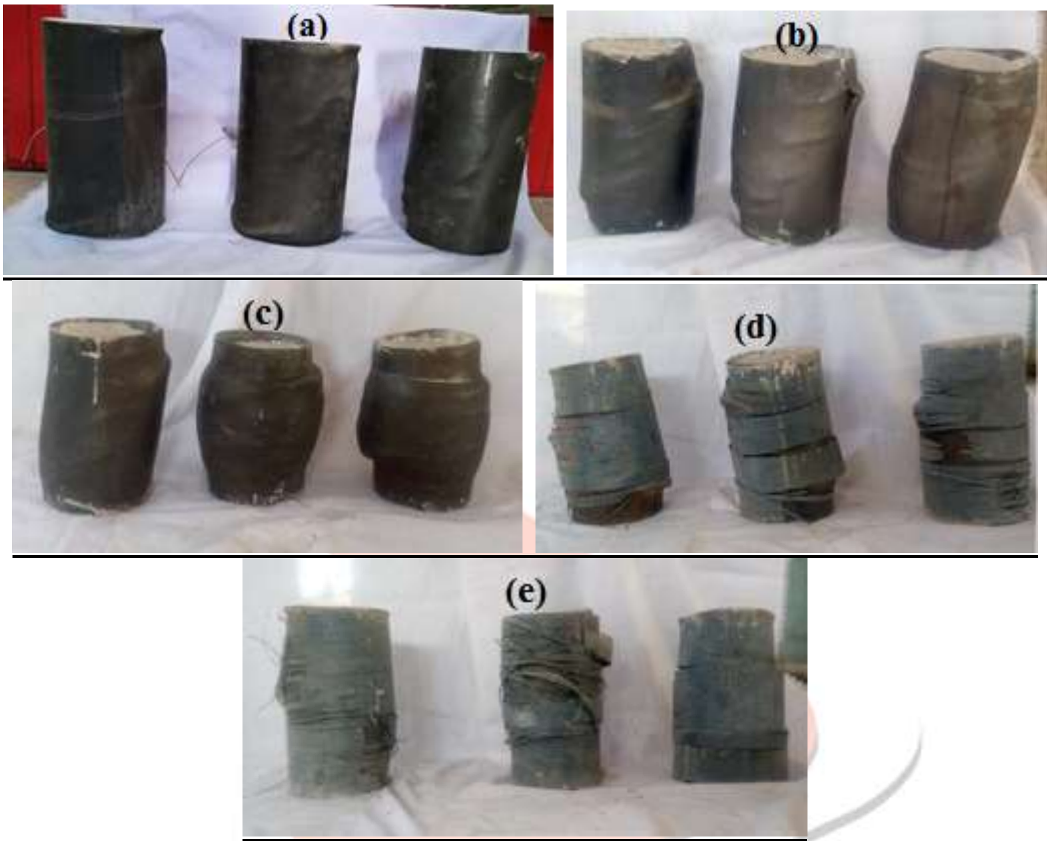

Figure 7. show the failure mode of the specimens. a) Set (1) CFST specimen's t2f0A, t2f0B*, and t2f0C*. b) Set (2) CFST specimen's t2f0A, t2f0B, and t2f0C. c) Set (3) CFST specimen's t3f0A, t3f0B, and t3f0C. d) Set (4) CFCFST specimen's t2f1A, t2f1B, and t2f1C. e) Set (5) CFCFST specimen's t2f2A, t2f2B, and t2f2C.*

Axial stress-strain curve for concrete core

Stress is calculated by dividing the load by the loaded area. For the set (1), the loaded area is the area of the concrete and the steel. So the contribution of the steel tube was subtracted to calculate the stress acting on the concrete only. For the rest of the sets, the axial stress calculated by dividing the load over only the concrete area while the strain calculated by the average values of the vertical strain gages induced at the mid-height of the specimens.

Set (1) specimens are shown in **Figure (8-a)**. The stress-strain curve for the concrete showed three main phases. The elastic phase, stress-strain relation almost linear, second when the steel tube yielded, and the relationship became curved. The stress at this stage is the maximum before decreasing again, and it is called the elastic-plastic phase. The final stage or the residual stage showed constant or slightly increase in stress-strain relation.

The specimens of sets (2, 3) in which only concrete is loaded as shown in **Figure (8-b, 8-c)**. The stress-strain curve for concrete core showed the same stages as of the set (1) in which the full section loaded with higher stresses. In the third stage, constant stress combined with massive strain increase. The steel gave a constant confining stress in the hoop direction after yielding, so no increase in the concrete stress declared.

The CFCFST specimens of sets (4, 5) only concrete is loaded and wrapped with FRP unidirectional jackets as shown in **Figure (8-d, 8-e)**. The Elastic stage showed linear stress-strain behavior. The elastic plastic stage, when the steel tube began to yield the stress-strain relation became curved. After the steel yielded, it provided constant confinement effect on the hoop direction, but the CFRP effect increases with the increase of loading. So the third stage (linear hardening) became linear again-with lower slop than the first stage- till the CFRP ruptures. The fourth stage (residual stage) begins with a sudden drop of the load due to CFRP rupture then remains constant or slightly increases. This behavior is as CFST third stage because when the CFRP jacket fails, the specimen behavior returns to its basic as CFST.

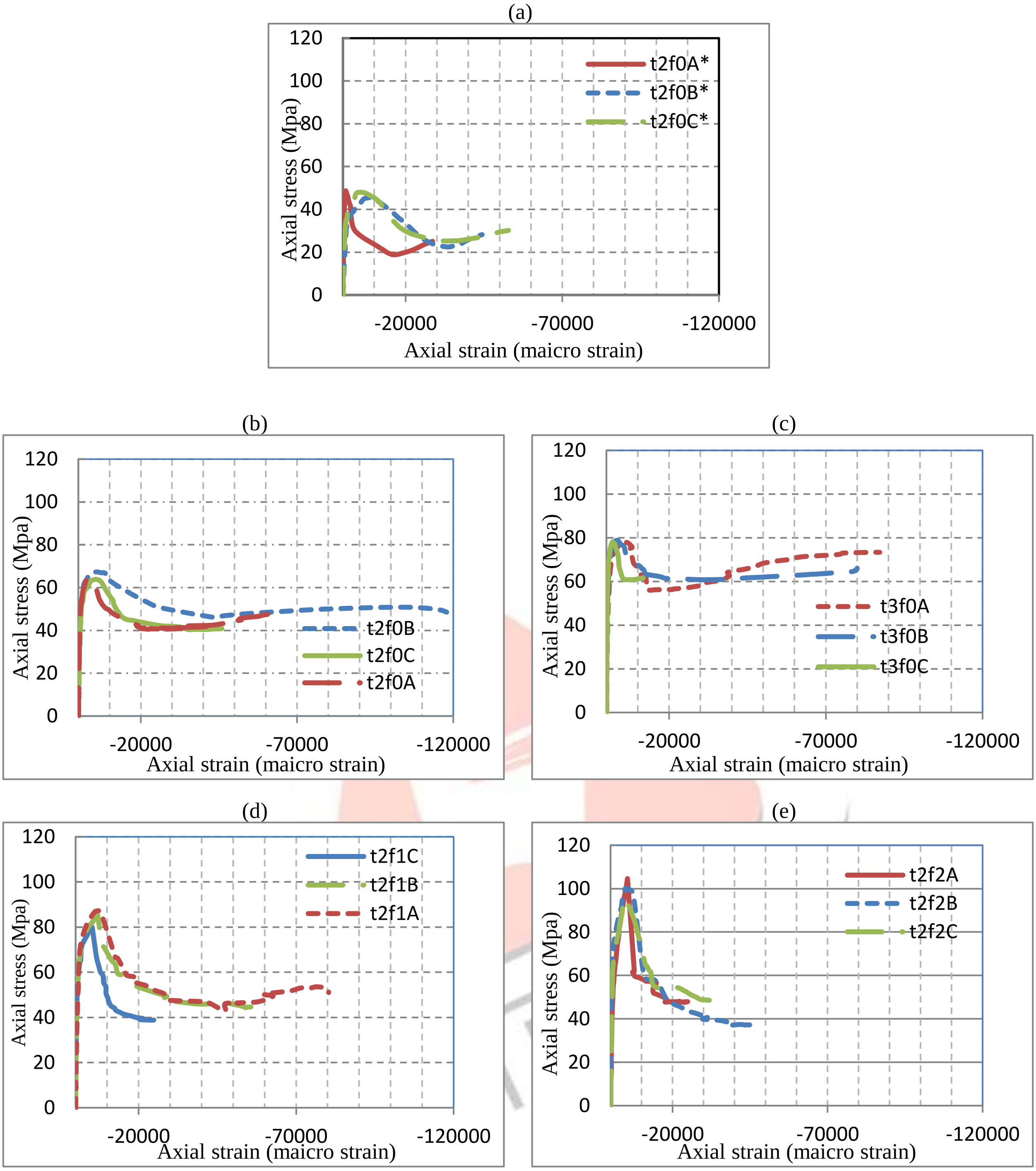

Figure 8. Stress strain curves for specimens. a) Full section loaded CFST specimen's t2f0A, t2f0B*, and t2f0C*. b) Concrete loaded CFST specimen's t2f0A, t2f0B, and t2f0C. c) Concrete loaded CFST specimen's t3f0A, t3f0B, and t3f0C. d) Concrete loaded CFCFST specimen's t2f1A, t2f1B, and t2f1C. e) Concrete loaded CFCFST specimen's t2f2A, t2f2B, and t2f2C.*

Comparison between full section loading and concrete core loading

The average concrete strength of set (1) t2f0A*, t2f0B*, and t2f0C* calculated and denoted as t2f0* then compared with the average concrete strength of the set (2) t2f0A, t2f0B, and t2f0C and denoted t2f0. As shown in **Figure (9-a)** for the CFST specimens with steel tube thickness equal to 2mm, the strength of concrete core increase about to 37% when the concrete core loaded without loading the steel tube. Generally, when the full section loaded, the steel tube acts as vertical and hoop reinforcement at the same time. However, when the concrete core only is loaded the steel tubes behave as transverse reinforcement only. In other words, any increase in the vertical stress will cause a decrease in the hoop strength of the steel tube.

The stress-strain behavior of specimens' t2f0* and t2f0 specimens are shown in **Figure (9-a)**. At the first elastic stage the same linear behavior with approximately the same stiffness slope except for the specimens where the steel tube was not loaded, the yield of the steel tube delayed until higher stress causing more extended elastic stage occurred. Also, the elastic plastic stage raised to higher stresses. The third stage (residual stage) also showed higher constant stress-strain behavior more than specimens which is entirely section loaded.

When only the concrete core is loaded, the maximum load carried by the CFST columns is higher 12% more than the load carried by the full section of the same thickness steel tube. In other words, the benefit of the steel tube became higher when the steel tube is used like lateral reinforcement only. **Figure (9-b)** shows the comparison between the axial load capacities for the previous two types of columns.

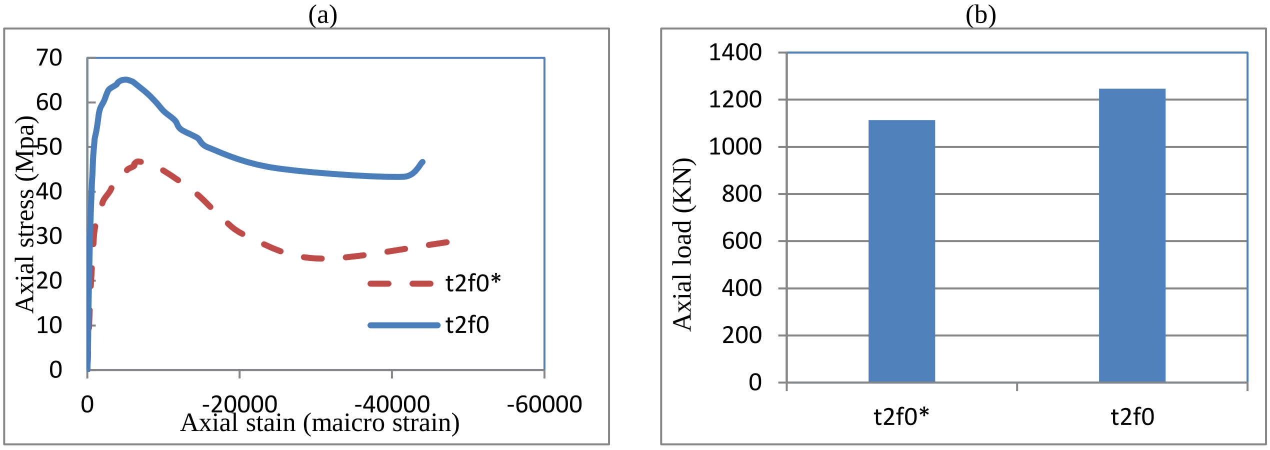

Figure 9. Comparison between full section loading and concrete core loading a) Stress strain curve for specimens' t2f0 and t2f0. b) Axial load capacities of specimens t2f0* and t2f0.*

Effect of steel tube thickness

The increase of the steel tube thickness enhances the strength of CFSTs due to the increase of the confining pressure produced by the steel tube. The stress-strain first linear stage (elastic stage) domain increased for higher stresses. Besides, the yield stress delayed up to higher loads when increasing the thickness. The second curved stage (elastic plastic stage) took place at higher stresses and the maximum strength increased. The third residual stage showed even higher residual stresses as predicted. The stress-strain curves for the average results of the set (2) t2f0 and the average results of the set (3) t3f0 are shown in **Figure (10)**.

The strength of concrete increased about 20% by increasing the steel tube from 2mm to 3mm thickness (about 50% increase of steel), so the overall weight also increased. If these types of columns subjected to an additional bending moment from crises like earthquakes or intense storms, the column may fail by an outward local buckling at the footing and beam-column connections. Such type of column will need the steel tube replaced, which would be difficult. For instance, wrapping CFRP jackets may be used at positions of stress concentration to enhance the performance of this type of columns. It does not require increasing the thickness of the steel tube or additional dead loads.

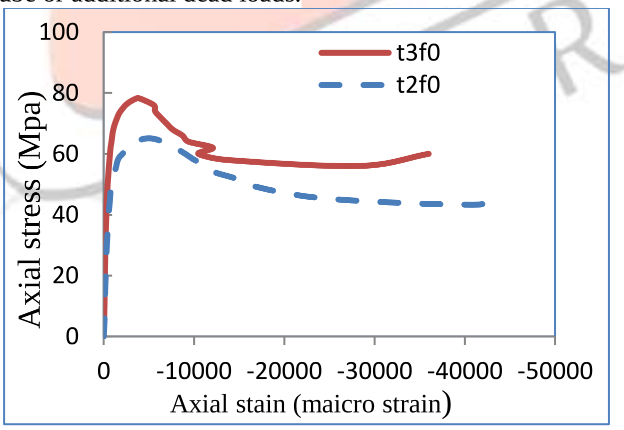

Figure 10. Stress strain curves for specimens' t2f0 and t3f0.

Effect of CFRP number of layers

Though the number of CFRP layers has an unnoticeable effect on the stiffness at the first elastic stage, it delays the yield of the steel for higher stresses. So the number of layers increases the domain of the elastic stage and delay the second elastic-plastic stage. The third stage of linear hardening gains higher stiffness with the increase of CFRP number of layers. The steel tube provides constant lateral pressure after yielding concurrently with the CFRP confining pressure raises linearly by loading until CFRP rupture. The fourth residual stage begins with a sudden drop that increases with the increase of the CFRP thickness while the residual stresses were decreasing as shown in Figure (11). The increase of CFRP number of layers is an excellent enhancement for the concrete compression strength as shown but the increase of the number of layers decrease the residual strength. This type of columns fails by rupture of CFRP. CFRP outer jackets may delay or prevent the local buckling of the steel tube especially in case of the additional moment due to seismic action or wind pressure. The repair of this type of columns can be made by replacing the CFRP jacket.

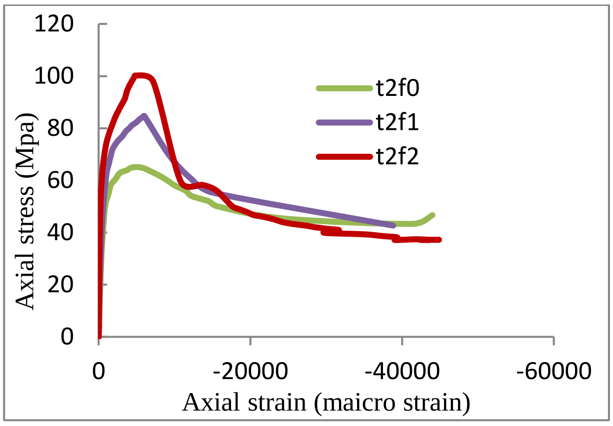

Figure 11. Stress strain curves of specimen's t2f0, t2f1, t2f2

Analytical prediction of axial capacity

The strength of the confined concrete is the most important factor to predict the axial load capacity of the CFCFST column. Mander [19] suggested an Equation (2) mainly depends on the lateral pressure introduced to the concrete core to calculate the confined concrete strength

$$f'_{cc} = f'_{co}\left[-2\frac{f_l}{f'_{co}} - 1.25 + 2.25\sqrt{1+\frac{7.9f_l}{f'_{co}}}\right] \quad \text{Eq.2}$$

In which f'_{cc} = the confined concrete strength, f'_{co}= the cylindrical concrete strength and f_l = lateral confining pressure can be calculated by Equation (3) as shown in **Figure (12)**.

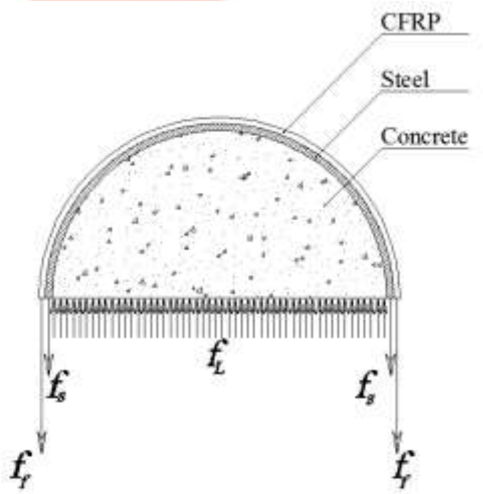

Figure 12. Lateral confining pressure f_l made by steel tube and CFRP layers.

$$f_l = \frac{2\,f_s\,t_s}{D} + \frac{2\,f_f t_f}{D} \quad \text{Eq.3}$$

In which f_s = steel yield stress, t_s= steel tube thickness, f_f = CFRP ultimate strength and t_f= thickness of CFRP layers.

Also a simplified Equation (4) made by lam 2002 [20] to calculate the confined concrete strength

$$f'_{cc} = f'_{co}\left(1 + k_1\frac{f_l}{f'_{co}}\right) \quad \text{Eq.4}$$

Where k1 is the confined effectiveness coefficient calculated by Xiao [21] and park [22] as shown in Equation (5).

$$f'_{cc} = \left(1 + 2.86\frac{f_l}{f'_{co}}\right) \quad \text{Eq.5}$$

Also researchers calculated the axial load using the constraint values for the steel and FRP because of the steel tube behaves under tri-axial complicated stresses in which the steel may be having different stress-strain behavior in each axis. The constraint factor takes the full axial capacity of the steel as ratio of the axial capacity of the concrete core. Yiyan Lu, Na Li and Shan Li (2014) [23] derived Equation (6) to calculate the axial capacity of CFCFST using constraint values.

$$N_u = (1 + 1.8\xi_s + 1.15\xi_f)f'_{co}\,A_c \quad \text{Eq.6}$$

Where N_u= axial load capacity, A_c = concrete area, and ξ_s is the steel constraint factor and calculated using Equation (7).

$$\xi_s = \frac{A_s f_y}{A_c f_c} = \frac{4t_s f_y}{Df_c} \quad \text{Eq.7}$$

Also ξ_f is the FRP constraint factor can be calculated using Equation (8).

$$\xi_f = \frac{A_f f_f}{A_c f_c} = \frac{4t_f f_f}{Df_c} \quad \text{Eq.8}$$

Yang Wei, Gang Wu, and Guofen Li [24] also used Equation (9) based on the constraint values of steel and constraint value of FRP the factors 1.27 and 1.28 was predicted as average value of a large number of specimens.

$$f'_{cc} = (1 + 1.27\xi_s + 1.28\xi_f)f'_{co} \quad \text{Eq.9}$$

For the current research the axial strength of each case calculated by the average value of three specimens. The results compared with equations derived by Yiyan Lu and the equation derived by Yang Wei as shown in **Table (5).**

Table 5

Specimen	Avg. experimental concrete stress	ξs	ξf	Yang Wei Equation	Yiyan Lu Equation	suggested Equation	$\frac{fexp}{fequ}$
t2f0*	46.75	.4	0	46.45	53.32	-	-
t2f0	66.1	.40	0	46.75	53.32	63.96	1.03
t3f0	77.07	.60	0	54.62	64.48	80.94	.952
t2f1	84.45	.4	.487	66.07	70.68	82.66	1.02
t2f2	100.21	.4	.974	85.40	88.043	101.36	.99

The results of the experimental study showed higher strength more than the predicted values due to the steel tube wasn't axially loaded directly but the friction and bond between the steel tube and the concrete core introduce the axial force to the steel tube. So the steel tube became more effective laterally and mad better confining effect. To predict the axial strength due to the increase of the axial stress in case of the steel tube is not axially loaded the following Equation (10) was derived:

$f'_{cc} = (1 + \xi_s k_0 + 1.28\xi_f)f'_{co}$ Eq.10

Where k_0 is the steel enhancement coefficient it was calculated by using the experimental results as coefficient multiplied by the steel constraint factor for Equation (11) From the average of the experimental results k_0=2.83

$f_{csf} = (1 + 2.83\xi_s + 1.28\xi_f)f'_{co}$ Eq.11

The equation agreed well with the test results with maximum erorr% $\cong \pm 5\%$

IV. CONCLUSION

The present experimental study investigated the behavior of CFCFST under axial loading introduced to the concrete core. Fifteen specimens were fabricated and tested to failure. Every three specimens represent the same type of column ending up with five cases. A developed analytical model was used to estimate the maximum axial stress for CFCFST. The following conclusion can be presented

1- CFST columns can resist higher axial loads when the load applied directly to the concrete core only compared with when the load introduced to the steel and concrete together.
2- CFRP outer jackets Added to CFST can be used as fuse element at positions of stress concentration especially when unexpected additional compression stresses occurred.
3- CFRP jacket delays or may prevent local buckling of the steel tube. Besides, it enhances the strength of the concrete core when more CFRP layer added, and the axial strain increases with increased CFRP thickness.
4- A new analytical model is proposed to predict the strength and the axial capacity of CFST columns when the concrete core is only loaded. The presented analytical model depends on the composite action between CFRP, steel tube, and concrete. The calculated results agree well within the range of presented experimental results.
5- The residual stage of the CFCFST shows lower strength with the increase of the thickness of the CFRP layers.

V. REFERENCES

[1]Lin-Hai Han, Wei Li, and Reidar Bjorhovde,"Developments and advanced applications of concrete-filled steel tubular (CFST) structures," Journal of Constructional Steel Research(2014) 100 211–228.

[2]Shafik Khoury, Ali Abdel-Hakam Aliabdo, and Ahmed Ghazy," Reliability of core test – Critical assessment and proposed new approach," Alexandria Engineering Journal (2014) 53, 169–184.

[3] Wei GU, and Hong Nan Li," Research in the Properties of The Concrete Filled Steel Tube Columns with Cfrp Composite Materials," Advanced Materials Research Vols 163-167 (2011) pp 3555-3559.

[4]Stephen P. Schneider, "Axially loaded concrete-filled steel tubes, " journal of structural engineering (1998)/1125

[5] Georgios Giakoumelis, Dennis Lam,"Axial capacity of circular concret filled tube columns," Journal of Constructional Steel Research 60 (2004) 1049–1068.

[6] P.K. Gupta, S.M. Sarda, M.S. Kumar,"Experimental and computational study of concrete filled steel tubular columns under axial loads," Journal of Constructional Steel Research 63 (2007) 182–193

[7] Fam, A. Z., and Rizkalla, S. H., "Behaviour of axially loaded concrete-filled circular fiber reinforced polymer tubes," ACI Structural Journal, (2001) V. 98, No. 3, May- June, pp. 280-289.

[8] QingJun Chen; Jian Cai; Mark A. Bradford, Xinpei Liu, and Yi Wu,"Axial Compressive Behavior of Through-Beam Connections between Concrete-Filled Steel Tubular Columns and Reinforced Concrete Beams," J. Struct. Eng.(2015), 141(10): 04015016

[9] Amir Fam, Frank S. Qie, and Sami Rizkalla,"Concrete-Filled Steel Tubes Subjected to Axial Compression and Lateral Cyclic Loads," J. Struct. Eng. (2004).130:631-640.

[10]Ming-Xiang Xiong, De-Xin Xiong, and J.Y. Richard Liew,"Axial performance of short concrete filled steel tubes with high- and ultra-high- strength materials," Engineering Structures 136 (2017) 494–510.

[11] Yang Wei, Gang Wu, and Guofen Li,"Performance of circular concrete-filled fiber-reinforced polymer-steel composite tube columns under axial compression," Journal of Reinforced Plastics and Composites (2014), Vol. 33(20) 1911–1928.

[12] Y. M. Hu; T. Yu; and J. G. Teng, " FRP-Confined Circular Concrete-Filled Thin Steel Tubes under Axial Compression," J. Compos. Constr. (2011).15:850-860.

[13]Yan Xiao; Wenhui He; and Kang-kyu Choi,"Confined Concrete-Filled Tubular Columns," J. Struct. Eng. (2005).131:488-497.

[14] Togay Ozbakkaloglu1 and Murat Saatcioglu,"Seismic Behavior of High-Strength Concrete Columns Confined by Fiber-Reinforced Polymer Tubes," J. Compos. Constr.(2006).10:538-549.

[15] Y.F. Zhang, J.H. Zhao, C.S. Cai," Seismic behavior of ring beam joints between concrete-filled twin steel tubes columns and reinforced concrete beams," Engineering Structures 39 (2012) 1–10

[16] Yunita Idris and Togay Ozbakkaloglu, "Seismic Behavior of High-Strength Concrete-Filled FRP Tube Columns,"J. Compos. Constr. (2013).17.

[17] Togay Ozbakkaloglu and Yunita Idris,"Seismic Behavior of FRP-High-Strength Concrete–Steel Double-Skin Tubular Columns," J. Struct. Eng. (2014).140.

[18] L'Hermite R. "Ide´es actuelles sur la technologie du be´ton.La Documentation technique du batiment et des travaux publics," (1955).

[19] J. B. Mander, M. J. N. Priestley,and R. Park,"Theoretical Stress-Strain Model for Confined Concrete," J. Struct. Eng.(1988), 114(8): 1804-1826.

[20] L. Lam, and J.G. Teng, "Design-oriented stress–strain model for FRP-confined concrete," Construction and Building Materials 17 (2003) 471–489.

[21] Xiao, Y., He. W. ,and Choi. K.K.,"Confined Concrete-Filled Tubular Columns," Journal of Structural Eng, ASCE, 131(3), (2005), pp. 488-497.

[22] J.W.Park., Y.K.Hong., G.S.Hong., and S.M.Choi.,"Experimental study on concrete steel circular tubes confined by carbon fiber sheet under axial compression loads," Journal of the earthquake engineering society of Korea, (2009), 13(5), pp 61-72

[23] Yiyan Lu, Na Li and Shan Li,"Behavior of FRP-Confined Concrete-Filled Steel Tube Columns, Polymers," (2014) 6, 1333-1349

[24] Yang Wei, Gang Wu and Guofen Li,"Performance of circular concrete-filled fiber-reinforced polymer-steel composite tube columns under axial compression," Journal of Reinforced Plastics and Composites (2014),33(20).