International Journal of Architectural, Civil and Construction Sciences
ISSN: 2415-1734
Vol:9, No:1, 2015

Behavior of Confined Columns under Different Techniques

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Abstract**—**Since columns are the most important elements of the structures, failure of one column in a critical location can cause a progressive collapse. In this respect, the repair and strengthening of columns is a very important subject to reduce the building failure and to keep the columns capacity. Twenty columns with different parameters is tested and analysis. Eleven typical confined reinforced concrete (RC) columns with different types of techniques are assessment. And also, four confined concrete columns with plastic tube (PVC) are tested with and with four paralleling tested of unconfined plain concrete. The techniques of confined RC columns are mortar strengthening, Steel rings strengthening, FRP strengthening. Moreover, the technique of confined plain concrete (PC) column is used PVC tubes. The columns are tested under uniaxial compressive loads studied the effect of confinement on the structural behavior of circular RC columns. Test results for each column are presented in the form of crack patterns, stress-strain curves. Test results show that confining of the RC columns using different techniques of strengthening results significant improvement of the general behavior of the columns and can used in construction. And also, tested confined PC columns with PVC tubes results shown that the confined PC with PVC tubes can be used in economical building. The theoretical model for predicted column capacity is founded with experimental factor depends on the confined techniques used and the strain reduction.

Keywords**—**Confined reinforced concrete column, CFRP, GFRP, Mortar.

I. INTRODUCTION

CONCRETE are compression members which transmit loads from the top to the lower levels, and then to the soil through the foundations. Since columns are the most important elements of the structures, failure of one column in a critical location can cause a progressive collapse of adjoining floors and may reach to collapse of the entire structure. In this respect, the repair and strengthening of columns is a very important subject to reduce the building failure and to keep the columns capacity.

Confinement is generally applied to members in compression, with the aim of enhancing their load carrying capacity or, in cases of seismic upgrading, to increase their ductility. Traditional confinement techniques rely on their either steel hoops or steel jackets for upgrading. Indeed, it is well known that increasing the confinement action enhances the concrete strength and ductility and, in addition, prevents slippage and buckling of the longitudinal reinforcement. This technique can also be useful in lap-splices zones [1].

The external confinement of existing reinforced concrete columns subject to improve their capacity and ductility has attracted many researchers. The repair and strengthening can be achieved using different techniques; there are conventional methods such as the using of external plain or reinforced concrete jacket [2] or steel jacket [3].

Confinement of reinforced concrete columns (or similar elements, like chimneys) significantly enhances the performance under axial load, bending and shear, because of the increase in concrete compressive strength, the increase in ductility, the increase in shear strength and the higher resistance against buckling of the steel reinforcement in compression.

The confinement of columns is achieved by means of internal lateral reinforcement (hoop or closed stirrups) or by external reinforcement (steel or FRP jackets). In the latter case, the confinement reinforcement can be provided either through external strengthening of existing columns, or as formwork that acts as structural reinforcement after construction of the columns. The main objectives of confinement are; a) To prevent the concrete cover from spalling. b) To provide lateral support to the longitudinal reinforcement. c) To enhance concrete strength and deformation capacities [4].

Silvia [5] reviews design guidelines for FRP confinement of reinforced concrete columns of noncircular cross sections, and states that: approach presented by current ACI committee [6] for compressive strength is based on the empirical formula originally developed by

Mander et al. proposed equation for steel confined concrete. Regarding CSA S806-02 guideline [7], the equation was obtained from experiments on cylindrical concrete confined under hydrostatic pressure by Richart [8] and Saatcioglu and Razvi [9]. The Concrete Society proposes a design-oriented model, developed by Lam and Teng [10]. And also, this model was calibrated against all the experimental data available at the time. The design recommendations provided by fib for columns of circular and noncircular cross sections are based on the model proposed by Spoelstra and Monti[11]. From previous discussion, it can be concluded that the calculated confined compressive strength only depends on the experimental data. In the paper, studied the different techniques can be used in construction and applied theoretical equation is done [12].

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II. Experimental Work and Test Results Analysis

The objective of this work is to study the effect of three different techniques of external confinement on the behavior of existing short circular reinforced concrete columns. These techniques are mortar strengthening, steel rings strengthening and FRP strengthening. The details of tested specimens are shown in Table I.

A. Experimental Work

All columns are circular columns of 120.0 mm diameter (*2r*) and 800.0 mm height (*h*) with concrete compressive strength equal to 25.0MPa. The clear concrete cover is 1.50 cm. Each column is reinforced with 6φ6 mm longitudinal reinforcing plain bars. The column spiral stirrups were provided of φ6 mm plain bars, and the pitches of stirrups were 5.0 cm.

The properties of composite materials (Mortar with admixture) for specimens CM1, CM2 are shown in Table I. The steel ring has 240MPa yield strength, and 360Mpa ultimate strength. The carbon fiber reinforced polymer (CFRP) has 3570MPa tensile strength (f_{fu}), 2346 GPa young's modulus (E_f), 0.13 mm thickness of one layer (t_f), and 0.015 ultimate tensile strain (ε_{fu}). The Glass fiber reinforced polymer (GFRP) has 1263.8MPa tensile strength, 576.3 GPa young's modulus, 0.3 mm thickness of one layer and 0.0214 ultimate tensile strain.

The electrical strain gauges are used to measure radial strains of the columns computerized universal-testing machine (U.T.M.) of 1000KN capacity was used to test the specimens as shown in Fig. 1. The load was incrementally applied to the column in specimen using compression state. Both the load (*P*) and corresponding deflection (δ) were recorded from the machine as well as the load-deflection (P-δ) curve was directly plotted. From the area of loading and the height of column (*H*) the stress-strain curve was drawn.

B. Ultimate Strength of Columns (f_{cc})

The load and strength capacities for tested columns, as measured experimentally are given in Table II. The external confinement increased the ultimate strength capacity of the circular columns which strengthened using epoxy mortar and rich cement mortar by 11.3% and 5.23%, respectively comparing to the reference column specimen capacity. The strength enhancements of the columns are 92.70%, 70.70% and 57.0% in case of using steel confining rings with spacing 50, 100, and 150mm, respectively comparing to the reference specimen strength. For the FRP group, the enhancement of the ultimate strength capacity of the circular columns strengthened with externally bonded fiber sheets are 205.1%, 137.2%, 42.5%, 30.3% and 82% for; fully confined with two layers of carbon fiber (CC2), fully confined with one layer of carbon fiber (CC1), partially confined with 50mm spaced between CFRP strips (CC3), partially confined with 100mm spacing with CFRP (CC4), and fully confined with one layer of GFRP (CG), respectively.

C. Crack Pattern and Mode of Failure

The crack patterns and mode of failures for tested specimens were shown in Fig. 2. The figure shows that the trends of the two specimens of mortar are similar. Specimen CM2 showed slight increase in load than specimen CM1 and they have nearly the same deflection at maximum load. In comparison with the reference specimen, the failure of both specimen CM1 and CM2 was to some extent sudden and brittle, while that of the reference specimen was more ductile and gradual. While the initial stiffness of the two strengthened specimens were higher than that of the reference specimen. Regarding to the ultimate strength capacity, strengthening using rich mortar and epoxy mortar enhanced the ultimate strength capacity by 5.23% and 11.30% respectively more than the capacity of reference column.

In the case of confined column, in general, the observed behavior of the confined columns was similar to the unconfined columns up to the peak load of unconfined columns. Increases in the lateral deflection of confined columns resulted in the concrete failing in compression and rupturing of the plastic tube. Tensile rupture of the tube was only caused by a significant increase in the lateral buckling beyond the peak load. The only signs of distress to the composite were the heard sounds and the large lateral deflections of the columns. Inspection of the concrete core after failure showed that the concrete had a highly fissured nature, but re-compacted under the high tri-axial state of stress.

Fig. 1 Tested set-up

TABLE I
TESTED SPECIMENS

Group	Code	Details of Tested specimens
Control specimens	CR	120mm diameters + 800 height and 6 bars 6mm vertical with yielding strength equal to 2400MPa and 5bars 8mm stirrups with 10mm pitch , and f_{co}=25MPa, E_c=22.5GPa,
Group One:	CM1	Confined concrete with rich mortar having f_{cu}=38MPa, and tensile strength=3.9MPa
Full confined with 2 cm mortar	CM2	Confined concrete with epoxy mortar having f_{cu}=44MPa, and tensile strength=4.5MPa
Group Two:	CS1	Inner spacing between ring =50mm
Partial confined with steel ring 50mm width and 3mm thickness	CS2	Inner spacing between ring =100mm
	CS3	Inner spacing between ring =150mm
Group Three:	CC1	CFRP two layers (full confined)
Confined with FRP, partial and full confined	CC2	CFRP one layer (full confined)
	CC3	One layer 50mm wide and 50mm spacing (partial confined)
	CC4	One layer 50mm wide and 100mm spacing (partial confined)
	CG	GFRP one layer (full confined)
Group four:	PVC-20	h=200mm, d=110mm, f_{co}=13.75MPA, ft_{pvc}=35MPa
The Confined with PVC tubes	PVC-30	h=300mm, d=110mm, f_{co}=13.75MPA, ft_{pvc}=35MPa
	PVC-60	h=600mm, d=110mm, f_{co}=13.75MPA, ft_{pvc}=35MPa
	PVC-90	h=900mm, d=110mm, f_{co}=13.75MPA, ft_{pvc}=35MPa

TABLE II
RESULTS OF CONFINED RC TESTED SPECIMENS

Group	Code	Pu (KN)	$P_{u, exp}$/Ac (MPa)	Ratio $P_u/P_{u,control}$	area under (P-δ)	δ-at $0.7P_u$	Ratio of increasing (δ-at 0.7 P_u)
Control	CR	210	18.59	100	0.55	0.10	100
G1	CM1	469	20.69	111.3	1.03	0.30	300
	CM2	444	19.56	105.2	1.11	0.34	340
G2	CS1	330	29.2	157.1	1.5	0.24	240
	CS2	339	31.75	170.7	1.8	0.26	260
	CS3	409	35.8	192.7	4.85	0.42	420
G3	CC1	501	44.09	237.3	3.5	0.44	440
	CC2	641	56.790	305.2	5.57	0.55	550
	CC3	300	26.490	142.5	2.2	0.29	290
	CC4	275	24.22	130.3	2.15	0.21	210
	CG	382	33.88	182.2	0.67	0.08	75

For the effect of slenderness ratio of confinement concrete column with plastic tube, it was found that the failure of confined specimens PVC-20 and PVC -30 were generally marked by sudden fracture of the plastic tube near the top and bottom edges of the tube. The pattern failure for specimens PVC-30 was shown in Fig. 2. Sounds heard during the early to middle stages of loading were attributed to the micro-cracking of concrete and shifting of aggregates. Snapping of the plastic tube could be heard near the end of the loading process. The failure of confined specimens PVC-60 and PVC-90 was generally due to shear cracks followed by yield of plastic tube.

D. The Stress Strain Curved.

The stress strain curves of group no.1 are plotted as shown in Fig. 3, the ultimate load was increased. The stress strain curves for group no.2 are plotted in Fig. 4. The steel rings strengthening increased the ultimate load and enhanced ductility for columns. Regarding to the cost of steel rings, the strains of specimen CS2 was 8.5% more than that of CS1. And the strain of specimens for CS3 was 61.9% more than that of CS2. From this concluded that, the increasing of column load stress and strain is not proportional linear with spacing between rings but in general by increasing the spacing between rings, the ultimate strength of confined column decreases.

Fig. 5 shows a comparison between stress strain curves for fiber specimen CC1 (one layer of carbon Fiber), CC2 (two layers of carbon fiber) and CC3 (8 strips of carbon fiber), CC4 (5 strips of carbon fiber), CG (one layer of glass fiber) and with the reference specimen (CR2).

From Fig. 5 and Table II, it is noticed the increase of load for tested specimen CC2 (2 layers CF) is 49.5% more than that of tested specimen CC1 (1 layer CF), while the difference in volume of the wrapping material between CC2 and CC1 is nearly about 100%, which represents increase of 100% in cost while the gain of capacity is 49.5%. The increase of load for specimen CC1 is 223% more than specimen CC3, while the difference in volume of strengthening material equals to 100%. Increase of load of specimen CC3 is 40% more than that of specimen CC4, while the difference in the volume of the wrapping material between these specimens equals to 60%. For the glass fiber, it has been found that the increase of load of tested specimen CC1 is about 71% more than that of specimen CG; the two tested specimens have the same area of strengthening covering the height of columns.

The energy absorption indicates the capability of an element to perform satisfactorily in the inelastic range. It calculated as the area under the load-displacement curve. Table II represents the energy absorption values for tested columns. As given in Table II, all kinds of confinements enhanced the energy absorption capacity for columns comparing to that of the reference specimen. The increase in energy was 87% and 101% for specimen CM1 and CM2, more than that of reference specimen, respectively. While for steel rings group the increase reached 173%, 227% and 782% for columns CS1, CS2 and CS3, respectively. FRP group showed increase in ductility reached 536% for CC1 (column confined with one layer of CFRP), and 913% for CC2 (column confined with two layers of CFRP. While it was 61% for CC3 (column confined with 8 strips of CFRP and 291% for CC4 (column confined with 5 strips of CFRP).

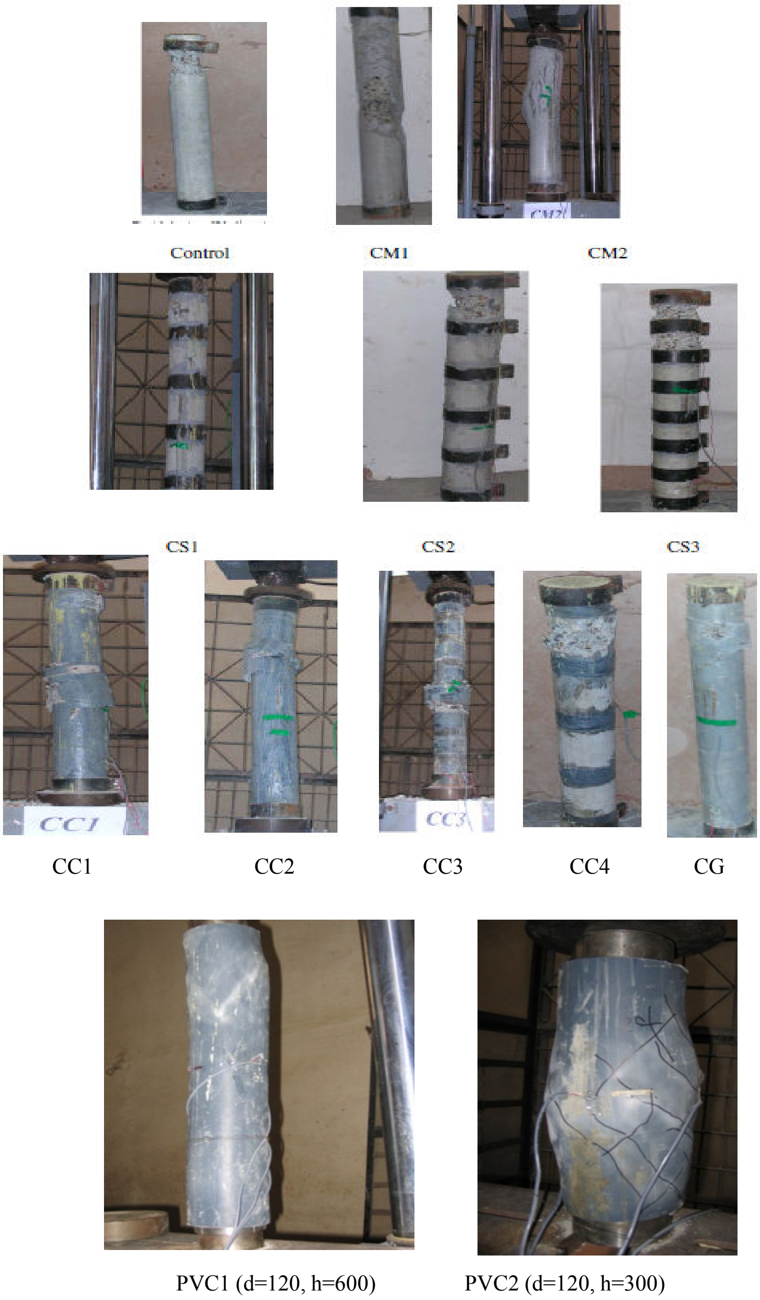


Fig. 2 Failure mode of tested specimens for confined columns

TABLE III
RESULTS OF CONFINED PC TESTED SPECIMENS

Group	Test specimen	*H/D*	P_{max} (Kn)	f_{co}*(Mpa)*	f_{cc} *(Mpa)*	f_{cc}/f_{co}	$f_{co(2)}/f_{co(H)}$	$f_{cc(2)}/f_{cc(H)}$
G4	PVC-20	1.82	293.0	*13.75*	*35.50*	2.60	1.00	1.00
	PVC-30	2.75	235.0	*11.9*	*29.50*	2.49	1.12	1.20
	PVC-60	5.45	227.5	9.5	26.70	4.10	1.46	1.33
	PVC-90	8.2	218.0	9.0	26.40	4.3	1.55	1.37

The column confined with one layer of GFRP (CG) showed a little increase in energy absorption (22%). It is found that, the high values of energy absorption were in the case of strengthening by 2 layers of carbon fiber. Partial confinement using CFRP gives lower values. The displacement at 0.7 of the ultimate load is considered in this study as a measure of displacement. Table II shows the displacement at 0.7 of the ultimate load for the tested columns. Moreover, as given in

International Journal of Architectural, Civil and Construction Sciences
ISSN: 2415-1734
Vol:9, No:1, 2015

Table II, confinement with mortar enhanced the deflection of RC columns by 240%, while the epoxy mortar enhanced the ductility by 200% comparing to that of control specimen. Steel rings group also enhanced the deflection by 140%, 160% and 320% for tested specimens CS1, CS2 and CS3, respectively. For tested specimen CC1 (confined with one layer of carbon fiber) the ductility increase was 340%, while for tested specimen CC2 (confined with 2 layers of carbon fiber) this percentage was 450%. It was 190% and 110% for tested specimens CC3 and CC4, respectively. Table III shows the tested results of confined concrete column with PVC tubes. The stress strain curved for tested column confined with PVC-tubes is shown in Fig. 6. For the radial strains, it was found that in the elastic zone there are no differences between strains. But, in the plastic zone, the radial strains begin deviations, especially for tested specimens SCC5-20, and SCC5-30. The deviation of radial strains can be credited to the positioning of gauges relative the cracks that begin appeared. The wide of crack increased quickly in the plastic zone. The slenderness ratio of confined column has significant effect on the plastic portion of the curve. By increasing the column slenderness ratio, the slope of curves in plastic part increase. In addition, the maximum stresses and strains decreased by increasing the slenderness ratio.

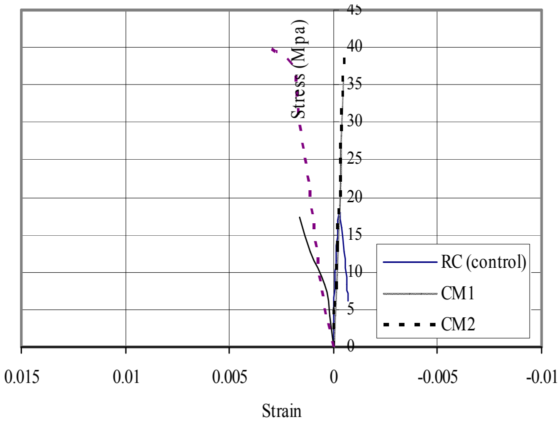


Fig. 3 Stress strain curves for the confined RC columns with Composite mortar

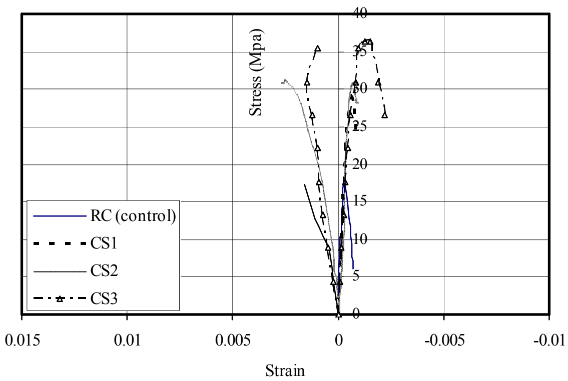


Fig. 4 Stress strain curves for the confined RC columns with steel ring

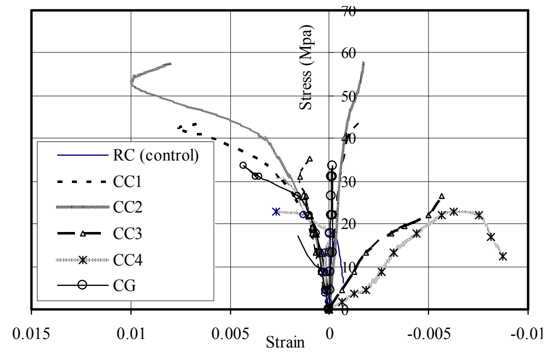


Fig. 5 Stress strain curves for the confined RC columns with FRP

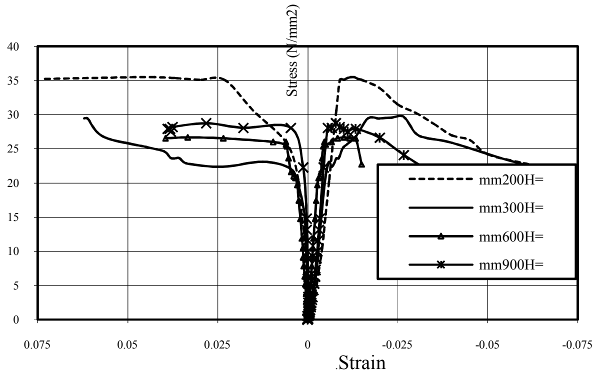


Fig. 6 Stress strain curves for the confined plain columns with PVC-tubes

III. Theoretical Study

In this part, the stages of change compatibility deformation between concrete core and FRP confined during loading are presented. Failure mechanism of confined concrete columns is illustrated. Moreover, the paper presents the proposal theoretical equation calculates the ultimate confined concrete strength (f_{cuc}). The by using the ultimate confined concrete the capacity Reinforced are four stages having different behavior according to the boundary condition between concrete columns can be calculated.

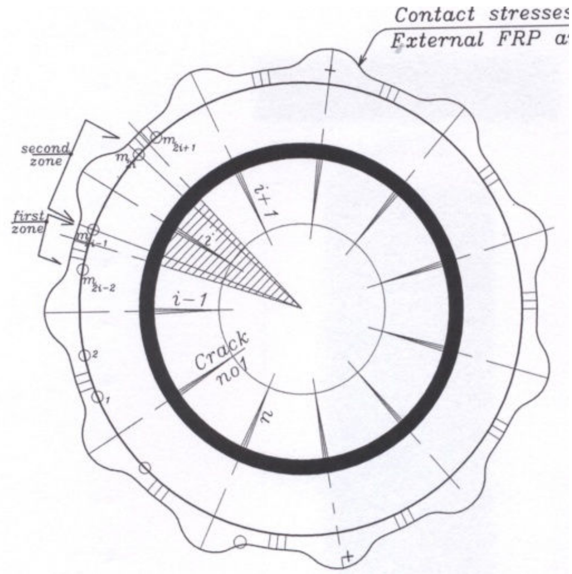
A. Stages of Change the Compatibility between Concrete Core and Confined FRP

During loading, there FRP sheet/tube and surface concrete core. The initial stage (the confinement action not fully acting); the elastic stage (the relation between stress and strain according to hook's low can be applied); the plastic-fracture stage (after cracking); the failure stage (after the peak load) [12], [13].

B. Failure Mechanism and Deformability

FRP confined concrete structures are made up of two materials with different characteristics, namely, FRP tube/sheet and concrete. Bazant and Xiang [14] illustrated the compression failure of unconfined concrete column. The crack model for confined concrete column, in macro scale, is shown in Fig. 7. The crack band model was widely used in practice for analyzing the distributed cracking and fracture of the concrete [13]-[16], and also, used for confined concrete failure. The concrete core is divided into number of units/segments and cracks equal to number n, and widths equal to b(r) as shown in Fig. 7 (a). The cracks appear when the tangent/or radial strain is more than the allowable tensile strain of the concrete. The real behavior of cracks, in the experimental tested specimens and by Cole, C. and Belarbi, A [17], as shown in Fig. 7 (b), agrees with the assumption crack patterns. The radial cracks propagate, and the FRP tube/sheet restrains internal cracks in the concrete core.

In the tangential direction, at the end of cracks, through contact surface between inner perimeter of FRP and the out perimeter of concrete core, the stresses in the FRP sheet/shell are not uniform. Two zones are as the follows:



(a) Proposal cracks distribution

(b) Real cracks distribution [12]

Fig. 7 Confined cracks model in macro scale

The first zone lies between point (m^{2i-2}) and point (m^{2i-1}), where *i* is the number of cracks.

The tangential/radial strains at perimeter of concrete core are less than allowable tensile strain. Thus, the crack width is equal to

$$w_{(r)} = b_{(r)}\varepsilon_o \tag{1}$$

where ε_o is the tensile tangential.

The second zone is between point (m^{2i-1}) and point (m^{2i}) [see Fig. 7 (a)]. The tensile strain is more than the allowable tensile strain of concrete and less than the allowable tensile strain of FRP sheet/tube. Thus, cracks appearance in concrete core, and the crack width through widths equal to $b(r)$ can be calculated as follows:

$$w_{(r)} = b_{(r)}\varepsilon_o + w_{c_{(r)}} \tag{2}$$

where $w_{c_{(r)}}$ is the crack width.

Fig. 8 shows the confined model in meso/or micro scale. At the contact surface between the concrete core and FRP sheet/tube, because of the allowable tensile strain between for FRP sheet and concrete is not equal, the differential movement happens. Subsequently two applying stresses occur, the first is a friction stress between the FRP sheet/tube and concrete core, and the second is the tensile strain of concrete core near the contact surface. At the same zone, the contact stress in the radial direction cause the third stress, as contact pressure (f_l) in the linear model. From this concluded that the applied concentrated stress near crack tips could be created. The concentration stresses causes additional acting stresses to the applied vertical stress.

C. Proposal Theoretical Model for Confined Concrete Strength

In the present studied, beside the general assumption of fracture mechanics and presented failure mechanism of confined concrete column, the additional assumption of confined concrete can be as the follows:

When the radial strain in the FRP tube is greater than the maximum allowable tensile strain of concrete, internal cracks propagate in the concrete core.

The confinement concrete columns delay formation a new cracks, and not prevent it.

The confinement of concrete column restrains development cracks.

The FRP tube/sheet restrains deformation with a force called resistance force of FRP sheet/tube (R_{frp}). In addition, the tensile stress in the un-cracked part is helping in the restraining with a force called resistance force of concrete (R_{ct}), the forces place is shown in Fig. 8. The horizontal applying tensile stress due to vertical stress causes tensile deformation of FRP tube/sheet with a force called acting force (C_{ver}).

Concentrated stresses besides the crack tips between point's m^{2i-1} and m^{2i} formed, and caused a force called acting force due to concentration stresses (C_{con}), as shown in Fig. 8. From equilibrium forces as shown in Fig. 8, the resistance forces (F_R) are equals to the acting forces ($F_{A.}$) as the following:

$$\sum F_{R.} = \sum F_{A.} \tag{3}$$

By calculating the resistance and acting forces, (3) can be as the following

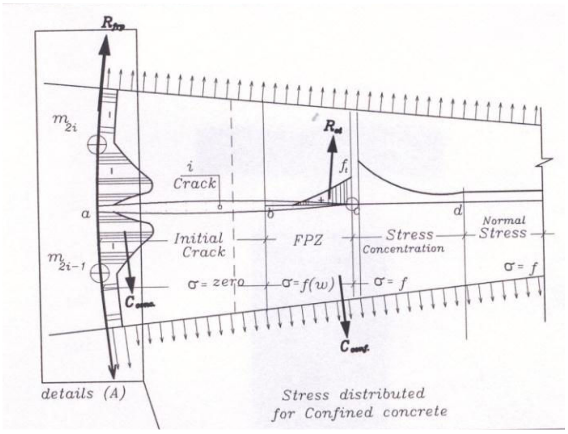


Fig. 8 Stress distribution, and resistance and acting forces position (Confined concrete model for meso/micro scale) [13]

$$f_{cc} = \left[\frac{0.8 \times f_{frp} \times t_{frp} + \frac{1}{2} \times \left[\frac{r \times E_{frp}}{E_c \times f_{frp}} \right] \times f_{tc}^2}{0.25 \times \left(\frac{t_{frp} \times E_{frp}}{r \times E_c + t_{frp} \times E_{frp}} \right) \times r + 3 \times t_{frp}} \right] \tag{4}$$

The fiber thickness (t_{frp}) for partial confined in equation can be equal to $\frac{nw}{h} t_f$ where w is the wide of strap of confined with thickness equal to t_f, and number n, for column height equal to h

D. Proposal Design Equation for Confined Plain Concrete Columns

For pain concrete column confined with PVC tubes, the proposal theoretical confined concrete strength can be as the following:

$$f_{cc,\exp} = f_{cc,theor} \times k \times f(\lambda) \tag{5}$$

where k is experimental factor, and can be assumed to calculate as the ratio between experimental confined strength to theoretical confined strength $f_{cc,\exp} / f_{cc,theor}$ for column slenderness ratio is equal to 3. And $f(\lambda)$ is experimental factor, takes the effect of column slenderness ratio (λ) into considerations.

Thus, the experimental factor (k), for specimens with slenderness ratio up to 3, as shown in Table III, can be approximate to 2.25. And for slenderness ratio more than 3, using excel program, and having best fitting, the factor $f(\lambda)$ can be approximate equal to $2.25 \times \frac{4.0}{\lambda^{0.88}}$.

Then, the final proposal equation for PVC tube confined concrete tested specimens was as the following:

$$f_{cc} = \begin{cases} \left[\dfrac{0.8 \times f_{frp} \times t_{frp} + \frac{1}{2} \times \left[\frac{r \times E_{frp}}{E_c \times f_{frp}} \right] \times f_{tc}^2}{0.25 \times \left(\frac{t_{frp} \times E_{frp}}{r \times E_c + t_{frp} \times E_{frp}} \right) \times r + 3 \times t_{frp}} \right] \times 2.25 & for \lambda \leq 3 \\ \left[\dfrac{0.8 \times f_{frp} \times t_{frp} + \frac{1}{2} \times \left[\frac{r \times E_{frp}}{E_c \times f_{frp}} \right] \times f_{tc}^2}{0.25 \times \left(\frac{t_{frp} \times E_{frp}}{r \times E_c + t_{frp} \times E_{frp}} \right) \times r + 3 \times t_{frp}} \right] \times 2.25 \times \\ \times \dfrac{4.0}{\lambda^{0.88}} & for \lambda \succ 3 \end{cases} \tag{6}$$

E. Proposal Design Equation for Confined Reinforced Concrete Columns

The Egyptian Code (ECP208) [14] recommends that, for reinforced concrete columns with existing closed stirrups; the ultimate load can be calculated as the following

$$P_u = \frac{0.67}{\gamma_c} f_{cuc} \times A_c + 0.67 f_y A_s \tag{7}$$

where P_u= the ultimate load capacity of the column; A_c= the net cross-sectional area of concrete; f_y= the yield stress of longitudinal steel reinforcement; A_s= area of longitudinal reinforcement; f_{cuc}=the ultimate confined compressive strength of concrete as a result of the use of FRP jacket.

The experimental confined concrete strength can be calculated as the following equation taken the experimental factor k is equal to unity.

$$f_{cuc} = \frac{0.67}{\gamma_c} \left[\frac{P_u}{A_c} - 0.67 f_y \times \rho_s \right] \times k \tag{8}$$

In general, the proposal theoretical confined concrete strength can be as the following:

$$f_{cc,\exp} = f_{cc,theor} \times k \tag{9}$$

where the factor (k) is experimental factor depends on the type of techniques and the reduction strain effect. By comparing between (6) and (7), and having best fitting, find that, the experimental factor depends on the techniques used. Form tested experimental results.

The experimental factor (k) is equal to as the following:

For confined mortar, the experimental factor (k) is equal to

$$k = 20 \times \left[\frac{t_f \times f_{fu}}{d \times f_{co}} \right]^{0.5} . \quad (10)$$

For partial confined with steel ring, the experimental factor (k) is equal to

$$k = 1.2 \left[\frac{t_f \times f_{fu}}{d \times f_{co}} \right]^{0.4} \quad (11)$$

For confined with FRP sheets, the experimental factor (k) is equal to

$$k = 0.5 \times \left[\frac{t_f \times f_{fu}}{d \times f_{co}} \right]^{0.5} \quad (12)$$

The ratios between experimental to theoretical values of theoretical equation are shown in Table IV.

TABLE IV
COMPARING BETWEEN THEORETICAL AND EXPERIMENTAL EQUATION

Group	Code	P_u (KN)	P_u/A_c (MPa)	$f_{cuc,exp}$	$f_{cuc,eq.}$	$f_{cuc,the\cdot}$	$f_{cuc,the}/f_{cuc/ex}$
Control	CR	21	18.6	10.04	10.04	---	----
Group one	CM1	46.92	20.6	11.38	2.55	12.64	1.11
	CM2	44.37	19.6	10.71	2.00	10.32	0.96
Group two	CS1	33.01	29.2	17.14	45.16	17.59	1.03
	CS2	35.87	31.7	18.81	43.85	19.16	1.02
	CS3	40.48	35.8	21.56	43.01	22.10	1.03
Group three	CC1	50.1	44.1	27.12	122.82	24.15	0.89
	CC2	64.11	56.7	35.56	138.56	38.54	1.08
	CC3	30	26.5	15.33	118.99	16.55	1.08
	CC4	27.5	24.2	13.79	121.54	13.80	1.00
	CG	38.24	33.82	20.23	120.91	21.49	1.06

The ratio between experimental to theoretical is between 0.96 to 1.1

IV. CONCLUSION

From the previous experimental tested results, it can be concluded that:

1. The confinement increases the column load capacity and ductility.
2. Confinements enhanced the energy absorption capacity for columns comparing to that of unconfined.
3. Carbon fibers increase the displacement capacity of circular reinforced concrete columns. It is obvious also that, the displacement capacity can be increased by increasing the number of layers.
4. The experimental results for PC column confined with PVC tube show that this type of construction can be used for economical building

From theoretical studied of mechanism of failure of FRP tube confinement concrete the following conclusions can be drawn:

1. The modes of failure change by varying slenderness ratios.
2. FRP tube/sheet confined concrete makes a restrain for cracks, and delays the cracks development but not prevents internal cracks.
3. The ultimate confined compression strength of reinforced concrete can be theoretically calculated by (8) taking the experimental factors (k) according to the techniques used.
4. The maximum confined compression strength depends not only on material properties ($E_{frp}, f_{frp}, E_c, f_{ct}$), but also on the size of specimen effect [h, r, t_{frp}].

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His research interest includes efficiency of the use of FRP (all types of fibers) in concrete and reinforced concrete elements. Specially, find theoretical model for prediction the strength of confinement concrete. Finally the nano-particles and used in construction one of new interesting directions.