NEW STEEL JACKET FOR RETROFITTING OF AXIALLY LOADED CIRCULAR REINFORCED CONCRETE SHORT COLUMNS

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ABSTRACT

A new steel jacket is proposed for retrofitting of circular reinforced concrete short columns. The installation of this jacket saves time, labor and cost in comparison to available jacketing systems. The assembly procedure would induce active confinement on the concrete column. Retrofitted column would be credited with the advantages of concrete-filled tubular columns. The jacket can be utilized for repairing and strengthening already existing columns and for new work regardless of column loading state; i.e. loaded or unloaded. A total number of 16 cylindrical short columns were tested under concentric axial loads until failure. The Concrete axial strength was increased in value more than twice. A design method is proposed.

Keywords: Steel-jacket, Circular-short-column, Reinforced concrete, Axial-strength, Retrofitting.

INTRODUCTION

For seismic design, most building codes have adopted the design concept of "strong column and weaker beam". This would discourage plastic hinging in columns. Retrofitting reinforced concrete column after damage caused by earthquake and/or to comply with new codes depends on jacketing the column. Several techniques and materials are presented in the literature. Jacketing of column using additional reinforced concrete is commonly used in Egypt and abroad. Experimental studies by Ersoy *et al.* [1] and Rodriguez *et al.* [2] showed improvement in strength and ductility when the jacket is made after unloading the column. However, "the jacketing was not very successful" when it is made for loaded columns. "The column in this case could carry only 50% of the axial load carried by a similar monolithic specimen" [1]. This technique is labor-intensive and time consuming and the improvement gained in strength is proportional to the increase in the column cross section and weight.

Advanced composite materials have been recently applied for jacketing reinforced concrete columns, [3 and 4]. It is used in the form of straps which are wrapped around the column in a continuous spiral and/or discontinuous rings. The results obtained indicate improvement of the confinement and hence in the strength and ductility. Another system for retrofitting reinforced concrete columns (5) uses a series of prefabricated E-glass fiber reinforced composite cylindrical shells. They are opened and clamped around the column in sequence. Adhesive is applied to bond the shells to each other and to the column to form a continuous jacket in the region of plastic hinge.

Steel jacketing has proven to be an effective measure to retrofit bridge columns for increased strength and ductility [6 - 9]. Circular cylindrical steel jackets are constructed in two half-shells slightly oversized for easy installation. The two halves are welded in site. The gap between the concrete column and jacket requires injection of grout as infill to enable composite action between the existing concrete and jacket. An increase in concrete compressive strength will result from the confining action of the steel jacket. For flexural retrofit, only the plastic hinge region of the column is retrofitted.

Most of the jacketing techniques presented above have been utilized for bridge columns. Relevant research related to building columns is that of concrete-filled steel hollow section composite columns. A review of experimental and analytical studies about the behavior of this type is presented by Shams *et al.* [10]. Concrete-filled tubular steel columns is credited with its high axial and flexural load carrying capacity, high shear resistance, greater critical load in buckling, large ductility and energy absorption in addition to saving of form work for the concrete core. This type, however can be utilized to new structures and to those already existing but having steel columns of hollow sections.

The authors propose a new steel jacket, described below, that can be used for retrofitting of circular reinforced concrete columns. The installation procedure of this jacket saves time, labor and cost in comparison to other jacketing systems. The assembly of the jacket would induce active confinement on the concrete column. Retrofitted column would be credited with the advantages of concrete-filled tubular columns. The jacket can be utilized for the repair and strengthening of already existing columns and for new work regardless of column loading state; i.e. loaded or unloaded. A total number of 16 cylindrical short columns were tested under concentric axial loads til failure. The obtained results are presented in tables and a design method is proposed.

THE PROPOSED STEEL JACKET

The jacket is cylindrical and is made of two half-shells of hot rolled steel sheet. It would be installed around the column as external continuous hoop over the full height of the column. For easy installation, a gap should be left in the length direction between the jacket and the ends of the column; i.e. beams and/or the footing. This would minimize any flexural enhancement which might cause excessive forces to develop in adjacent members, Priestley *et al.* [7 and 8]. The ends of the two shells at one side are prepared for welding on site. At the other side, they are over-lapped and prepared with the connection details shown in Figure 1-a. Two steel angles of the same size, grade and length are used. The angles have the same length of the shells. They are prepared to have bolts' holes of the same diameter, position and number along their length. They are adjusted so that the centers of the bolts' holes coincide. The angles are then fillet welded to the shells. High strength bolts are inserted in the coinciding holes to connect the two angles together and hence the two shells to form one jacket. Tapered washers are used under both bolts heads and nuts. Having installed the jacket, the bolts are tightened by hand to snug position. Finally, torque wrench is used to tighten each bolt to a defined load. As a result of this, the two angles and the ends of the shells, would move towards each other. The distance V left between the two angles should be just enough to allow for this movement and for any tolerance required for the manufacture process. Tightening the bolts would cause tension stresses in the steel jacket and hence lateral active radial pressure on the concrete column. When axial load is applied to the column, the concrete would be subjected to triaxial stress. The active radial pressure in addition to the confinement provided by the steel jacket is expected to increase the concrete strength. When welding in site is not allowed or more lateral pressure is required to apply on the concrete column, two connections of the same details as described above are provided at the two sides of the shells, as seen in figure 1-b.

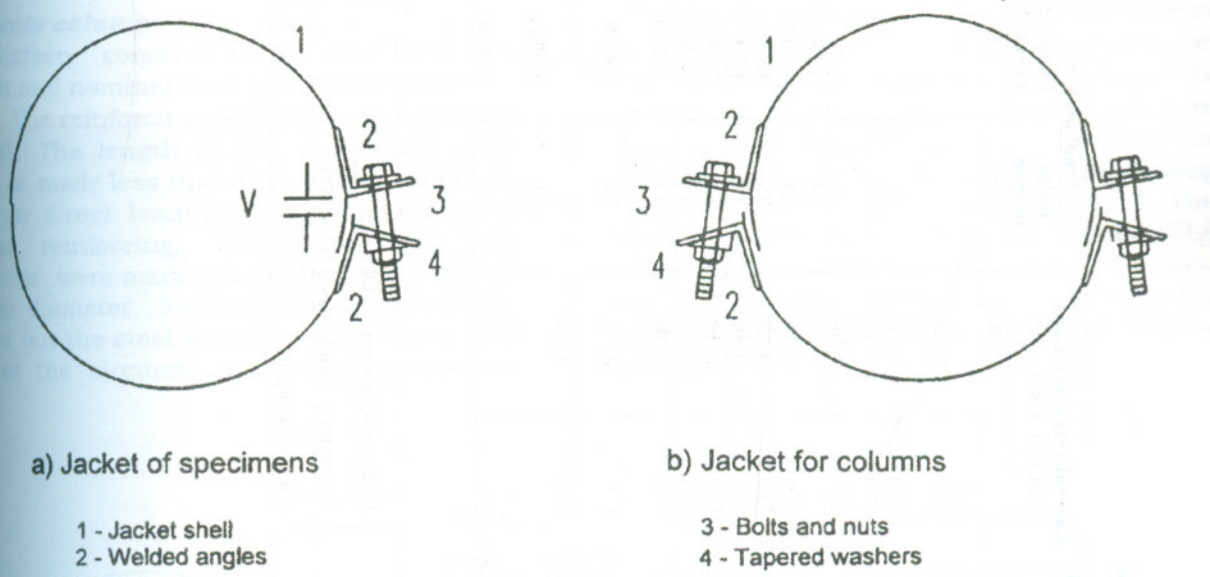


Figure 1 Details of the proposed steel jacket cross section

EXPERIMENTAL PROGRAM

A total number of 16 columns were tested under concentric axial load. The dimensions and details of the specimens are presented in table 1. To verify the use of the proposed steel jacket for new work, strengthening and repairing of already existing columns, the tests and the results were divided into four groups. In group A, no steel jackets were used and concrete columns were loaded to failure. In group B, the columns were confined using the steel jacket. Bolts were tightened to defined values as shown in Table 1. Specimens JNRA35^s AND JORA35^s were loaded only to 80% of their expected ultimate loads and the tests were stopped. The specimens were investigated visually. In group C, the steel jacket was used but the bolts were not tightened. Only the first and last bolts were hand tightened to avoid premature failure. The concrete columns were loaded to 600 kN. At this load, the bolts were tightened to a torque of 35 N.m and the loading procedure continued until failure. In group D the concrete columns were loaded to failure first and then retrofitted. Firstly, the jacket was installed but the bolts were not tightened except the first and last bolts. The jacket was used at this stage to avoid the problems of installing the jacket after spilling of the concrete cover and buckling of the longitudinal reinforcing steel bars. Axial load was applied til failure. The bolts were then tightened and the retrofitted column was loaded again til failure.

Test Specimen

Steel Jacket

For this study, there was no need to manufacture the steel jacket as two half shells. The concrete column is short and not connected at its ends to other structural elements. A one piece jacket will cause no problem in the installation procedure and will not affect the obtained results. Ten steel cylinder jackets having a length of 600 mm were made from hot rolled steel sheets of 2 mm thickness and grade 37. The jackets were prepared with the connection details shown in Figure 2. Two angles of size 40x40x4, length 590 mm and grade 37 were prepared to have bolts holes of 13 mm diameter at a pitch of 60 mm along their length. Fillet welds of equal leg and size 3.0 mm were made at the positions shown in Figure 2 to join the steel angles to the shell. Tapered washers were placed under both bolts heads and nuts to compensate for the unparallel surfaces to which the bolts heads and nuts react.

Table 1 Details of experimental specimens

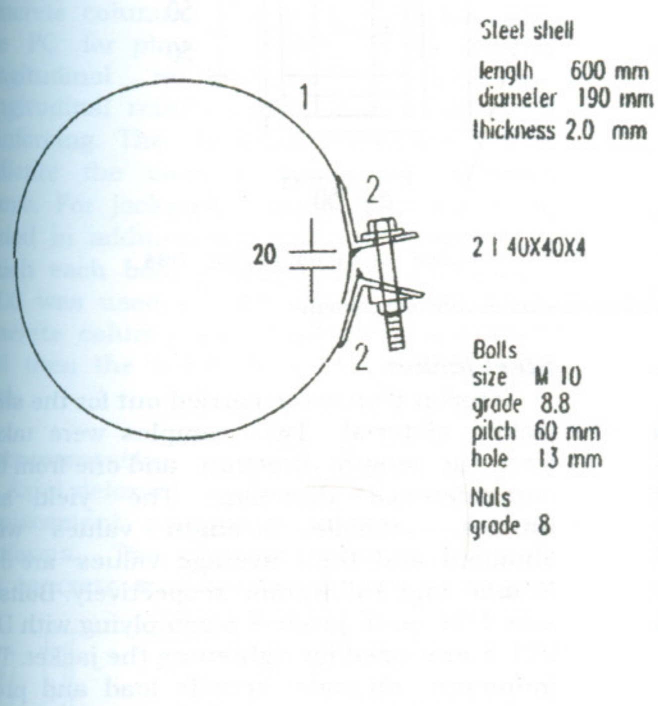
Specimen			Concrete strength	Longitudinal reinforcement deformed bars			Lateral reinforcement diameter = 8.0 mm		Steel jacket	
No.	Group	Name							Bolts torque	Induced tension
			N/mm^2	No.	Diameter mm	ρ %	s	ρ %	N.m	kN
1	A	PCB	18.13	-	-	-	-	-	-	-
2		NRB	18.13	6	12	2.34	170	0.48	-	-
3		PCA	20.42	-	-	-	-	-	-	-
4		NRA	20.42	6	12	2.34	170	0.48	-	-
5		ORA	20.42	6	16	4.1	170	0.48	-	-
6		OSA	20.42	6	12	2.34	50	1.63	-	-
7	B	JPCA14	20.42	-	-	-	-	-	14	6784
8		JNRA14	20.42	6	12	2.34	170	0.48	14	6784
9		JNRA35	20.42	6	12	2.34	170	0.48	35	16960
10		JNRB14	18.13	6	12	2.34	170	0.48	14	6784
11		JNRA35^s	20.42	6	12	2.34	170	0.48	35	16960
12		JORA35^s	20.42	6	16	4.1	170	0.48	35	16960
13	C	JNRA35-L600	20.42	6	12	2.34	170	0.48	35	16960
14		JORA35-L600	20.42	6	16	4.1	170	0.48	35	16960
15	D	NRA*	20.42	6	12	2.34	170	0.48	-	-
		JNRA35*	-	-	-	-	-	-	35	16960
16		PCA*	20.42	-	-	-	-	-	-	-
		JPCA35*	20.42	-	-	-	-	-	35	16960

NOTES:

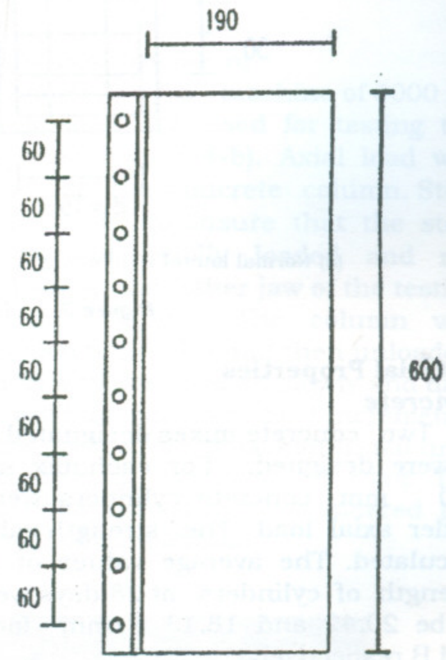
- Concrete columns:	Diameter = 190 mm	length = 600 mm	length / Diameter = 3.15 < 5
- Steel jacket :	Thickness = 2.0 mm	Diameter = 190	Diameter / Thickness = 95 < 100
- Bolts :	Size M10	Grade 8.8	pitch = 60 mm

Concrete column

Sixteen concrete columns of 600 mm length and nominal diameter of 190 mm were cast. The reinforcing details are presented in Table1. The length of the reinforcing steel bars is made less than the column length so that no direct loading was applied on it. For lateral reinforcing, hoops of 150 mm diameter were made using plain steel bars of 8 mm diameter. No anchor was made in the hoops but the steel was over-lapped over 100 mm at the circumference. Concrete cover of 20 mm was provided at the circumference of the column. Hoops were concentrated at the top and bottom regions of the column to prevent premature failure, as seen in Figure 3-a. In only one specimen OSA, prepared for comparison purposes, hoops were used along the specimen as shown in Figure 3-b. The steel jackets were used as forms. The columns were cast vertically. After 24 hours, the forms were removed and the concrete were cured for 28 days in 100% relative humidity.

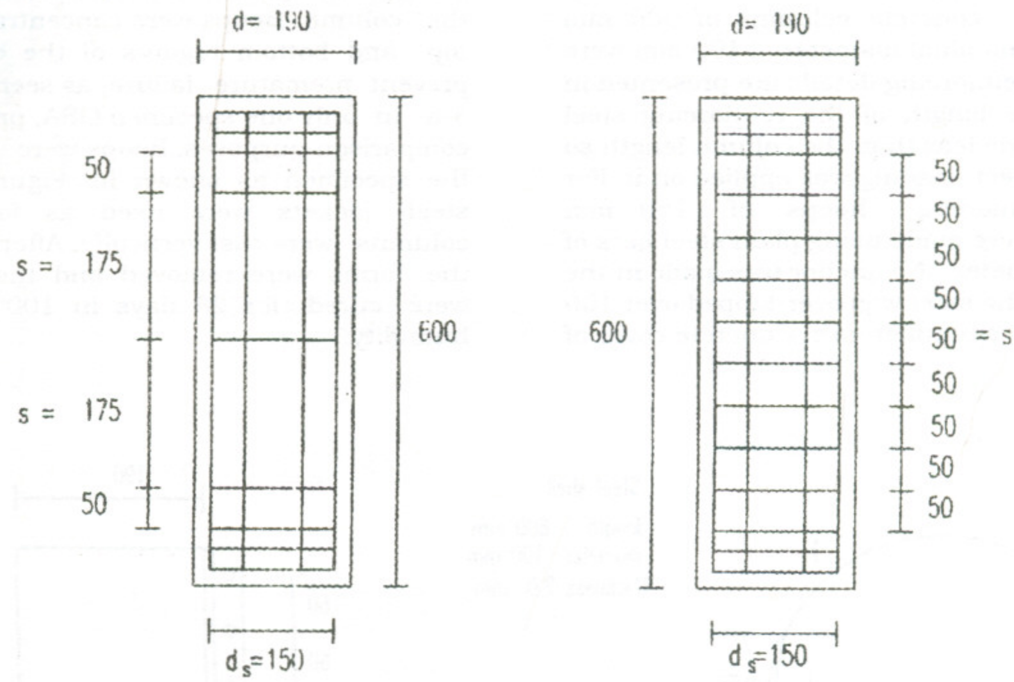


(a) Cross section



(b) Elevation

Figure 2 Details of the steel jacket used in the experimental program



(a) Normal lateral reinforcing

(b) Over lateral reinforcing - OSA

Figure 3 Details of reinforcing steel in concrete columns.

Material Properties

Concrete

Two concrete mixes designated as A and B were designed. For each mix, six 150 X 300 mm concrete cylinders were tested under axial load. The strength values were calculated. The average values of the axial strength of cylinders at 28 days were found to be 20.42 and 18.13 N/mm^2 for mixes A and B respectively.

Reinforcement

Tension testes were performed on three samples of each bar diameter. The average values of the yield and ultimate strengths were calculated and presented in Table 2.

Table 2 Material properties of reinforcing steel

Bar Diameter mm	Yield Strength N/mm^2	Tensile Strength N/mm^2	Elongation %	Notes
12	415.16	671.0	26	Deformed
16	414.34	697.49	18.7	Deformed
8	282.53	448.31	28	Plain

Steel jacket

Tension tests were carried out for the steel jacket material. Two samples were taken from the length direction and one from the circumference direction. The yield and ultimate tensile strength values were obtained and their average values are 340 N/mm^2 and 485 N/mm^2 respectively. Bolts of size M10 and grade 8.8 complying with DIN 931 were used for tightening the jacket. The minimum ultimate tensile load and proof load for this size and grade of bolt specified in ISO 898 - 1 [11] are 46400 N and 33700 N respectively. Nuts of grade 8 complying with DIN 934 were used

Specimen Assembly

Concrete columns were externally confined at the top and bottom regions by steel collars. This prevented premature failure of the column at these regions, [12 and 13]. For retrofitted columns, the steel jacket was installed and the bolts were tightened to a defined torque using torque wrench, (see Table 1). The tightening procedure was carried out in the following order. After tightening bolt n , bolt n-1 was retigthened. Bolt n+1 was then tightened. This process was carried out for all the bolts in a sequential order to avoid bolts relaxation. This procedure was repeated to insure the inducing of the required tension force in all the bolts. Plaster of paris was then used at both ends of the concrete column to eliminate uneven top or bottom surfaces. Each specimen was tested 48 hours later to allow for nut relaxation to occur.

Specimen Designation

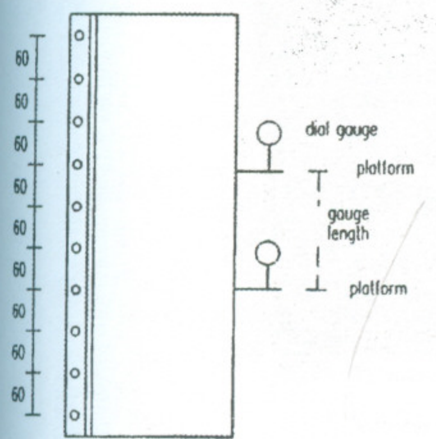
Three characters were used to designate concrete columns. The first two characters are PC for plain concrete, NR for normal longitudinal reinforcing, OR for over longitudinal reinforcing or OS for over lateral reinforcing. The third characters is A or B to indicate the concrete grade as described above. For jacketed columns, the letter J is added in addition to the torque value in N.m which each bolt was tightened to. The term L600 was used for the cases at which the concrete column was loaded first to 600 kN and then the bolts were tightened at that load.

Instrumentation

Axial deformations were measured by two displacement gauges reacting against two platforms. The platforms were attached to the concrete surface through 40 X 40 mm square openings. They were made in the steel jacket specially for this purpose at 85 mm above and below the mid height of each column, (see figure 4-a). This would make a gauge length of 170 mm. At nearly the ultimate load, concrete cover spills. The platforms become no longer fixed to the concrete surface and the displacement measurements in this stage were not taken into account. This did not allow the recording of the descending part of the load deformation relationship. However, the technique is suitable for the jacketing procedure utilized in this study and was used before, (14 and 15).

Test Setup and Procedure

A Universal testing machine of 3000 kN loading capacity was used for testing the columns, (see Figure 4-b). Axial load was applied only to the concrete column. Steel shims were used to insure that the steel jacket was not axially loaded and not reacting against the other jaw of the testing machine until failure. The column was initially loaded to 200 kN and then unloaded. The column was then loaded slowly and data were recorded at selected load increments. The loading procedure was continued until significant drop in load was recorded. The maximum load recorded is considered the ultimate load of the column.



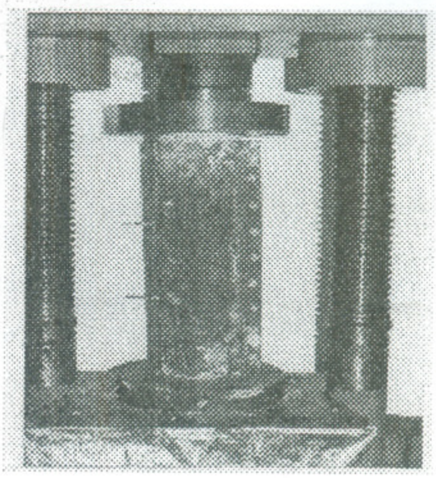


Figure 4 Specimen in the testing machine

Failure Modes

The failure of PCA was characterized by concrete crushing at mid height of column, as seen in Figure 5-a. Figure 5-b shows that PCB failed in a diagonal plane. Reinforced concrete columns failed in similar manner. Nearly at the ultimate load, the concrete cover was spilled off. The loading procedure was continued until the concrete core was destroyed and the longitudinal steel bars had buckled, as seen in Figures 6 and 7.

Retrofitted columns of groups B and C failed in a similar manner.

Local buckling occurred at the sides of the 40x40mm openings made in the steel jacket, as seen in Figure 8. The concrete cover did not spell off except at these openings. Vertical cracks were observed in the concrete column in the length direction. This was concentrated at the region of the jacket connection, as seen in Figure 9. No concrete crushing was observed. The column was still intact in one unit but the concrete become fragile. JPCA14 failed after testing when removing the steel jacket. This caused the fracture shown in figure 10. Figures 11-a and 11-b show the cracks observed in columns JNRA35s and JORA35s respectively. Most of the cracks were found in the side of the jacket connection rather than in the other side.

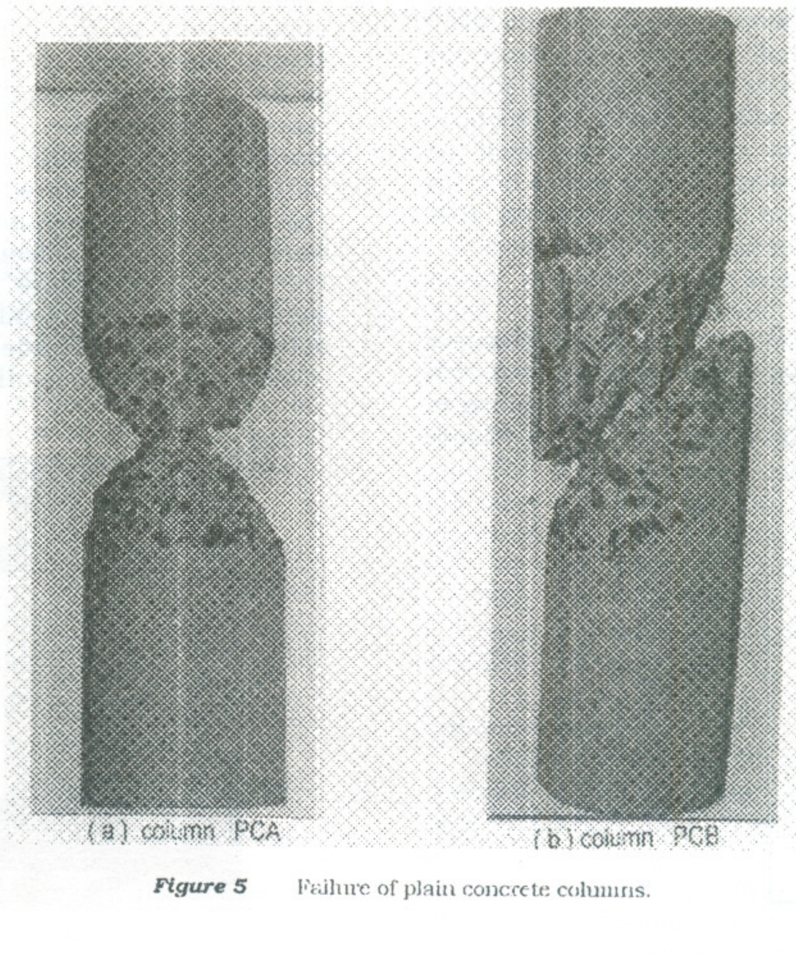


Figure 5 Failure of plain concrete columns.

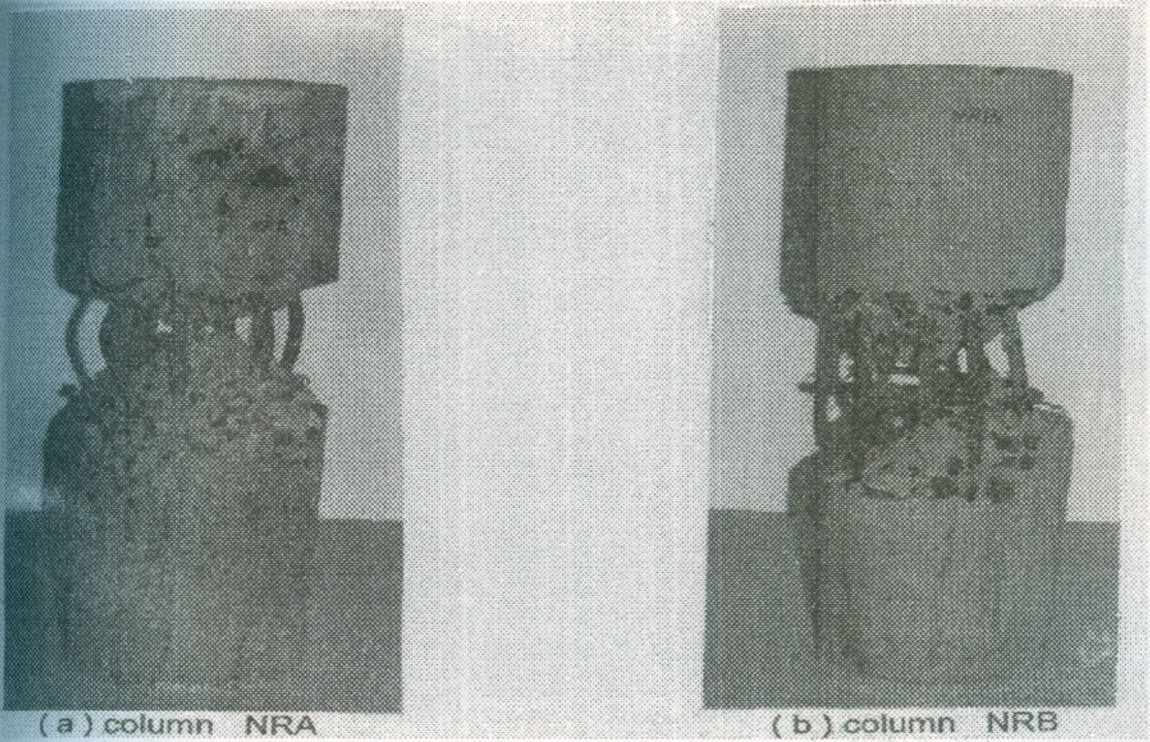


Figure 6 Failure of reinforced concrete columns, NRA and NRB

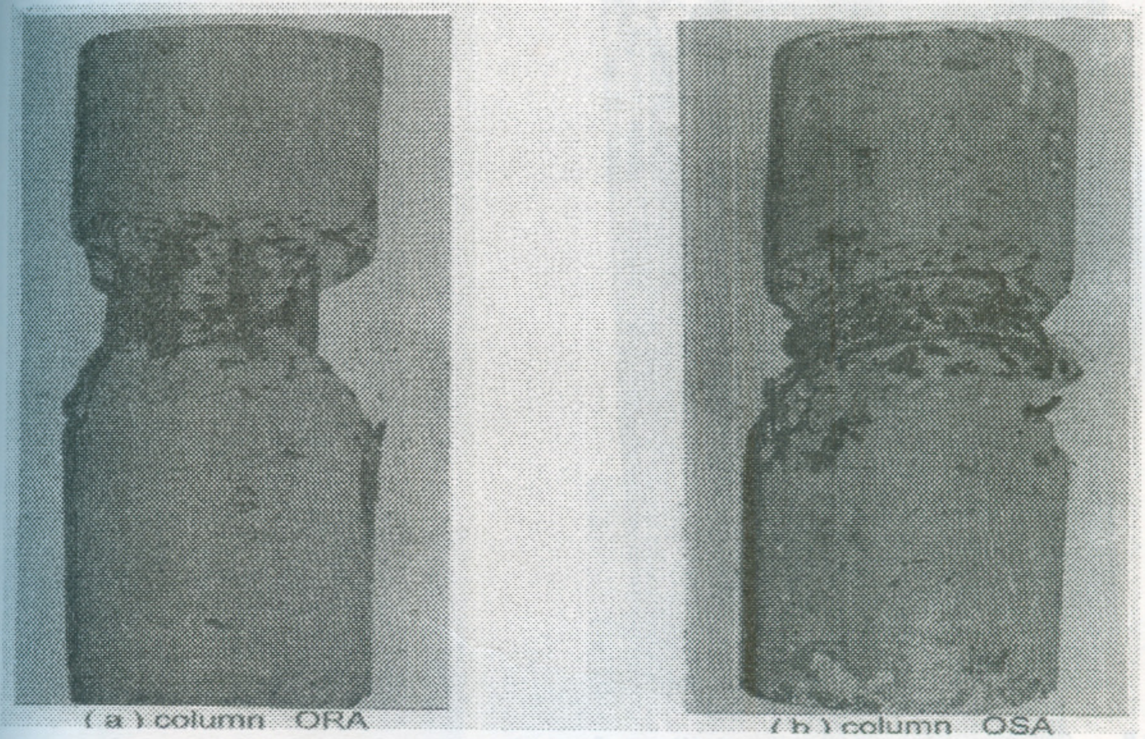


Figure 7 Failure of reinfororced concrete columns, ORA and OSA.

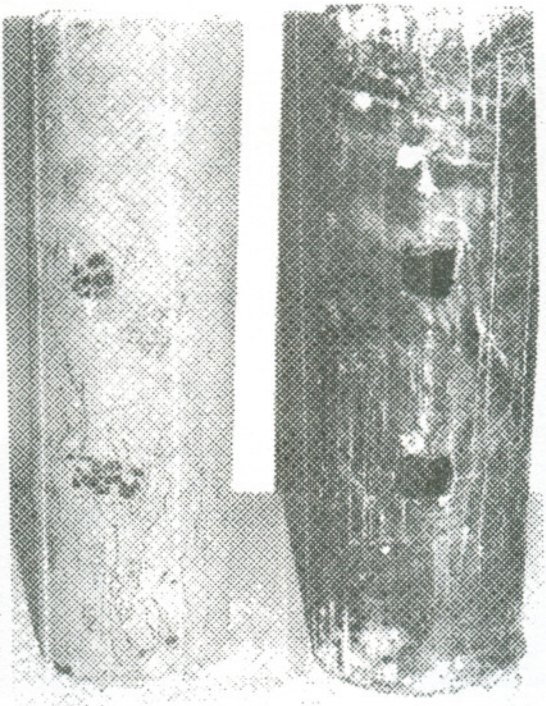


Figure 8 Column JNRB14 at failure.

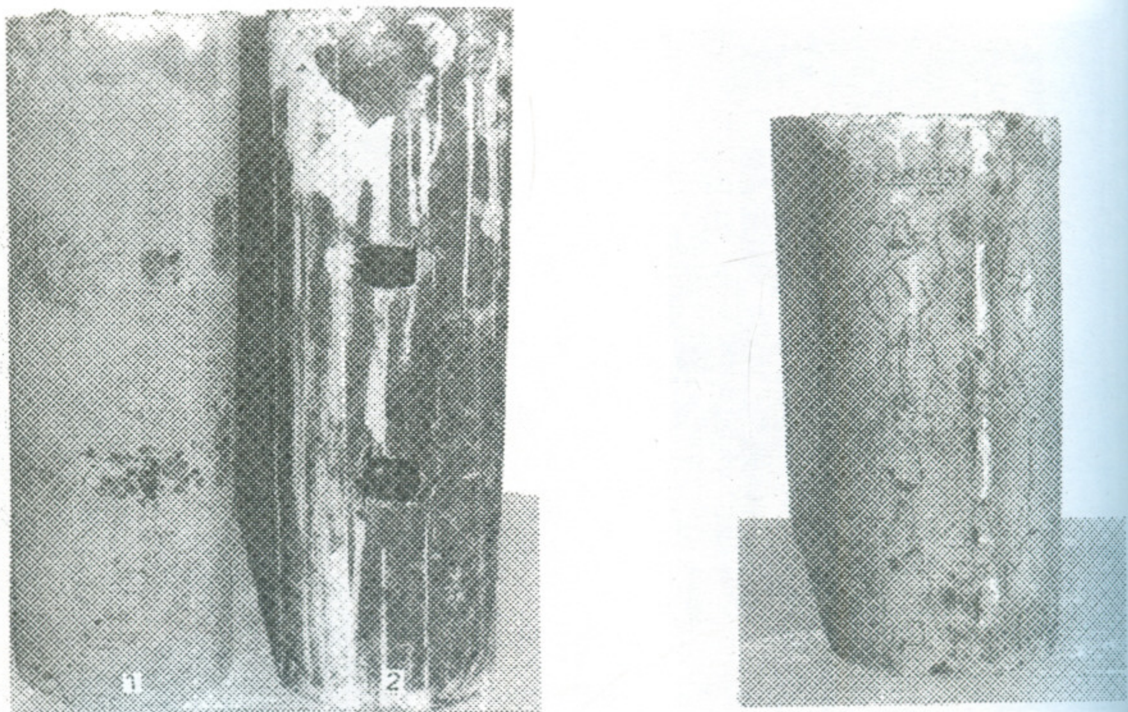


Figure 9 Column JORA35 – L600 at Failure: (1) part of concrete cover were spilled off; (2) Local bucking occurred in the steel jacket; (3) Vertical cracks at the region of the jacket connection;

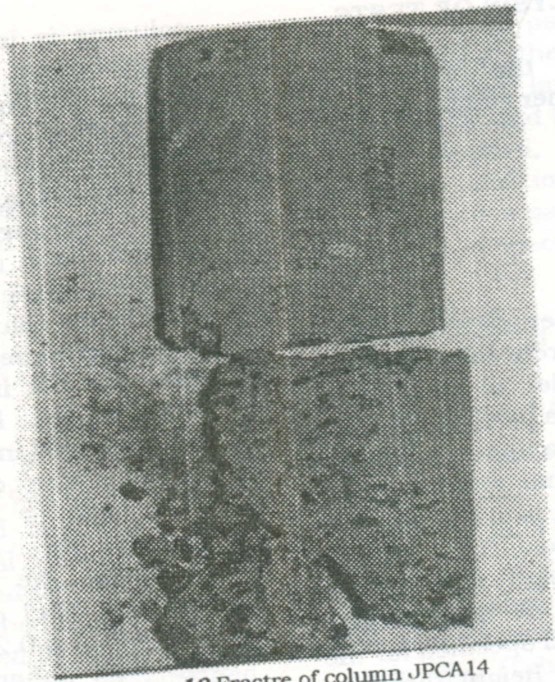
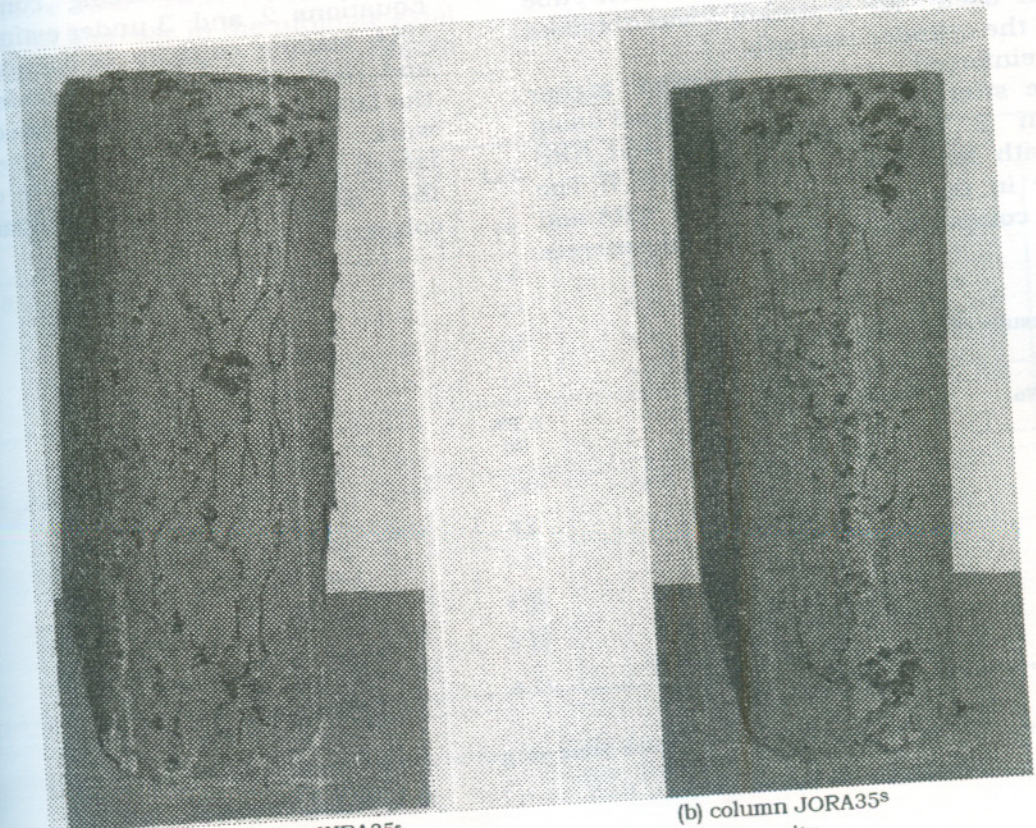


Figure 10 Fractre of column JPCA14



(a) column JNRA35[s]

(b) column JORA35[s]

Figure 11 cracks at 80% of ultimate loading capacity.

RESULTS AND ANALYSIS OF TESTS

Concrete Columns

Table 3 includes the experimental ultimate loads P_t and other computed values determined as follows:

$$\Delta P\,\% = 100\ (P_t - P_c)\ /\ P_c \quad (1)$$

$$P_o = \alpha\, f_c\ A_c + A_s\ f_y \quad (2)$$

$$P_w = f_{cu}\ A_c + 0.44\ A_s\ f_y \quad (3)$$

Where P_c is the experimental ultimate load of unreinforced concrete column having the same concrete grade of the column considered and ΔP % is the percentage of the enhancement in column loading capacity due to the use of longitudinal reinforcement. Equation 2, with $\alpha = 0.85$, is of the ACI 318 Committee, [16]. Equation 3 is of the Egyptian Code for the design of reinforced concrete structures [17]. It is a working stress method equation and specified for the design of columns having height/diameter ratio not less than 5. It is used in this study for comparison purpose. The concrete cube strength used for equation 3 was obtained as 1.25 the cylinder strength.

Unreinforced columns showed higher ultimate strength than that obtained using Equation 2. The results of NRA and NRB agree with Equation 2, (Table 3 - col 4). The increase in the ultimate loads of NRA and NRB in comparison to unreinforced concrete columns is in the range of 17%. This limited enhancement is less than that produced when adding the yield load of the longitudinal reinforcing steel to the ultimate load of unreinforced column. Equation 2, in this particular case, under estimates and over estimates the concrete and steel contributions respectively. This refers to the low volumetric ratio of the confining lateral steel in the columns.

The increase in the longitudinal steel ratio did not improve the ultimate load of column ORA. Equation 2 over estimated the ultimate load in this case (Table 3 - col 4 and 5). The value obtained using Equation 3 for this case is not consistent with those obtained for NRA and NRB. In contrast, increasing the volumetric ratio of lateral confining steel ρ =1.64% and reducing the ratio s/d_s to 0.26 as column OSA, displayed large load enhancement. This is due to the satisfactory performance of circular hoops at these lateral reinforcing conditions [13]. Equations 2 and 3 under estimated column OSA loading capacity in comparison to NRA and NRB. They do not include the effect of the lateral confining steel. More experimental work is needed to decide whether or not lateral confining steel should be expressed in the design equations similar to spiral columns.

Table 3 Results of concrete columns – group A

Specimen No.	Name (1)	P_t kN (2)	Δ P% (3)	P_t / P_o α= 0.85 (4)	P_t / P_o α= 1.0 (5)	P_t / P_w (6)
1	PCB	610	-	1.396	1.186	-
2	NRB	719	17	1.0	0.903	2.64
3	PCA	693	-	1.408	1.12	-
4	NRA	809	16.7	1.046	0.94	2.79
5	ORA	795	14.7	0.8	0.74	2.09
6	OSA	990	42.8	1.28	1.15	3.42

Jacketed Columns

Figures 12 and 13 compare between the relationships obtained of the applied loads and axial strains of different column specimens. Table 4 includes the experimental ultimate loads and other computed values determined as follows:

$$P = P_t - A_s\ f_y \quad (4)$$

$$f_{cc} = P / (A_c - A_s) \quad (5)$$

$$f_{cc} = f_c(-1.254 + 2.254[1 + (7.94 f_L / f_c)]^{1/2} - 2 f_L / f_c) \quad (6)$$

$$R = (f_{cc} - f_c) / f_L \quad (7)$$

Equations 4 and 5 were used to calculate the experimental values of concrete axial strength in jacketed columns f_{cc}. The corresponding lateral radial pressure causing confinement f_L is calculated from Equation 6. This equation was described by William and Warnake [18] and adopted by Mander *et al.* [19] in his theoretical model for confined concrete. Concrete axial strength was enhanced more than twice (Table 4 - col 4). This caused an increase in the ultimate axial load of jacketed columns in comparison to their counter parts of Table 3, column 3. Tightening of the bolts in the jacket would induce active lateral radial pressure σ_a, (see Equation 8 and Figure 14-a). When an axial load is applied, the confining action of the jacket in addition to the hoops, if it exists, would induce passive lateral radial pressure, σ_p and σ_s respectively. Equations 9 to 11 and Figures 14-b and 14-c illustrate the relationships between the different components of induced radial pressure at failure.

$$\sigma_a\ S_b\ d/2 = T/2 \quad (8)$$

$$2\ f_{ys}\ A_{ss} = \sigma_s\ s\ d_s \quad (9)$$

$$f_L = \sigma_s + \sigma_a + \sigma_p \quad (10)$$

$$T1/2 = S_b(\sigma_a + \sigma_p)\ d/2 \quad (11)$$

The value of f_L is the actual affecting radial pressure while σ_p is the net value applying on the concrete column. The values of the ratio R, (Table 4 - col 5) obtained using Equations 4 to 7 are in the range found by Richart *et al.* [20]. This applies to all the columns except that of JORA35-L600.

Table 4 Results of retrofitted columns.

Specimen No.	Specimen Group	Specimen Name	P_t kN	$X = P_t / P_t^*$	f_{cc}/f_c	$(f_{cc} - f_c)/f_L$	σ_j/σ_{jy}	T_b/T_y	P_t/P_D
		(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
7	B	JPCA14	1280	1.85	2.21	4.44	0.94	1.14	1.06
8		JNRA14	1640	2.02	2.4	4.12	0.98	1.19	1.05
9		JNRA35	1590	1.97	2.32	4.26	1.14	1.38	1.14
10		JNRB14	1433	2.0	2.31	4.26	0.79	0.96	0.98
11		JNRA35^s	1300	1.61					
12		JORA35^s	1670	2.1					
13	C	JNRA35-L600	1664	2.05	2.45	4.05	1.28	1.55	1.19
14		JORA35-L600	1997	2.51	2.7	3.65	1.59	1.92	1.26
15	D	NRA*	687	0.85	0.72				
		JNRA35*	1450	1.79	2.06	4.69	0.91	1.09	1.04
16		PCA*	490	0.7	0.85				
		JPCA35*	1275	1.87	2.2	4.45	0.97	1.18	1.25

Note :
P_t^* experimental ultimate load of concrete column having the same details of the considered column but not jacketed (from Table 3)

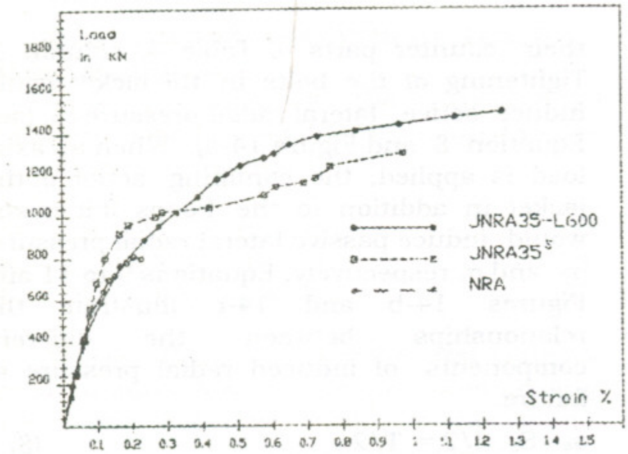


Figure 12 Applied load - axial strain relationship of columns NRA, JNRA35^s and JNRA35-L600.

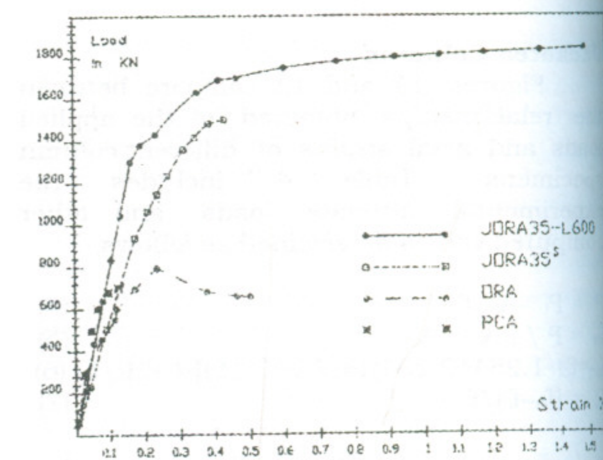


Figure 13 Applied load - axial strain relationship of columns ORA, JORA35^s, PCA and JORA35-L600.

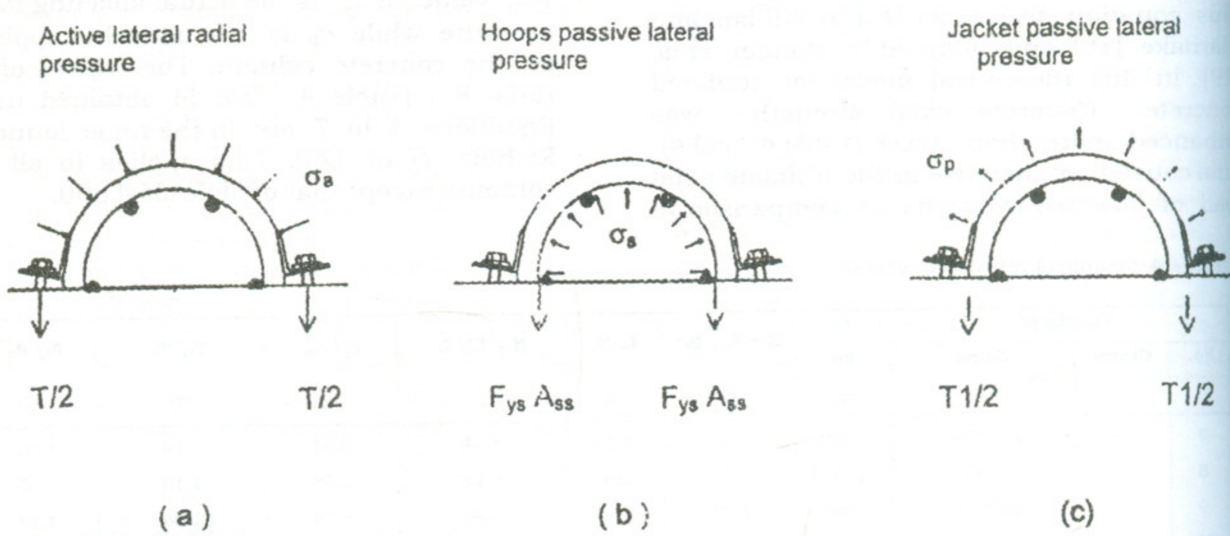


Figure 14 Confining action due to (a) bolts tightening, (b) hoops and (c) steel jacket.

Increasing the tightening degree of the bolts, as the case of JNRA35 in comparison to JNRA14, would insure good contact between the jacket and the concrete column and increase the induced active lateral pressure. The net lateral pressure applied on the concrete column would also be reduced at failure. This is beneficial when the lateral pressure value is approaching the concrete strength. However, the forces induced in the bolts and the shell of the jacket are increased, (see Equation 11). The change in concrete axial strength as the case of JNRB14 did not affect the ratios f_{cc}/f_c and P_l / P_t * (Table 4 - col 3) to a significant level. This showed the success of using the jacket to confine columns made of low axial strength concrete.

For the cases of retrofitting already loaded columns, Equation 8 would be modified as follows:

$$(\sigma_a + \sigma)\, S_b \; d/2 = T/2 \qquad (12)$$

where σ is the lateral radial pressure produced due to the initial applied axial load.

This would reduce σ_a value and hence the forces induced in the bolts and the shell. This is expected to cause higher ultimate load in comparison to non loaded columns as the case of JNRA35-L600. Column JORA35-L600 showed relatively high ultimate axial load and strength. The authors find no explanation to this result at the time being. The retrofitting of already failed concrete columns, group D, gave nearly the same results of their counter parts of group B. This result agrees with the findings of Gardener *et. al.* [21], (specimens 31 and 32).

Failure Mechanism

Failure of jacketed column would initiate due to yielding of the bolts and/or the shell. The relatively large elongations in the bolts and/or redial strains in the shell would cause reductions in the confining lateral pressure and hence the concrete axial strength. The concrete column would be cracked as described earlier. The induced forces at failure in the bolts and shell of the jacket are calculated using Equation 13. The obtained results are compared to bolt proof load and yielding load of the jacket shell and presented in columns 6 and 7 of Table 4.

$$\text{force in bolt} = \sigma_j \; t_j \; S_b = (T+T1)/2 \qquad (13)$$

Design Procedure

The design is made assuming that the column is unloaded and yielding would occur in the bolts before the jacket shell. Further, the bolts would be tightened to a degree that insure good contact between the jacket and the concrete column inducing active lateral radial pressure. In practice, it is recommended to inject a thin layer of strong filling material between the concrete column and the steel jacket before tightening the bolts. This would minimize the effect of column circumference irregularity and ensure good contact between the column and the jacket. The data required for the design procedure are 1) the required enhancement of the column loading capacity, 2) the strength of standard concrete cylinder, 3) the column geometrical cross section details and 4) the longitudinal and lateral reinforcing steel details. The value of f_L is calculated using equations 4 to 6. The value of P_t would be the required loading capacity. The values of σ_s , σ_a, σ_p and T1 are calculated by Equations 8 to 11. The shell thickness and the bolts size, grade and pitch would be defined according to the value obtained from Equation 13. This method is used to calculate the design ultimate loads of the columns considered in the experimental program. The calculated values are in acceptable range of the experimental results, (Table 4 - col 8). This does not agree with the results of group C due to neglecting the radial stress σ .

CONCLUSIONS

The proposed steel jacket may be used for 1) new work and 2) strengthening and 3) repairing of already existing columns. Tightening the bolts of the jacket after injecting a filling material between steel jacket and column would insure good contact between the jacket and the concrete column and induce active lateral pressure. The confining action of the jacket in addition to the hoops, if exists, would induce passive lateral radial pressure when axial load is applied. Concrete axial strength was enhanced more than twice. This showed increase in the ultimate axial load of jacketed columns in comparison to their counter parts of unjacketed columns. Increasing the tightening degree of the bolts would reduce the net value of applying lateral radial pressure on the concrete and increase the induced forces in the bolts and the shell of the jacket. Failure of the column would initiate due to yielding of the bolts and/or the shell. The occurred elongation and/or redial strain would reduce the confining lateral pressure on the concrete and hence its axial strength. The use of the jacket with already loaded columns showed relatively superior behavior. The jacket showed its success in retrofitting already failed columns.

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NOMENCLATURE

A_c	Gross sectional area of concrete column.
A_s	Cross sectional area of longitudinal reinforcing steel.
A_{ss}	Cross sectional area of hoop.
D	Diameter of concrete column.
d_s	Diameter of concrete core ,hoop to hoop centers.
f_c	strength of standard concrete cylinder.
f_{ca}	allowable axial stress of concrete.
f_{cc}	axial strength of confined concrete.
f_L	Lateral radial confining stress on concrete.
f_y	yield strength of reinforcing steel.
f_{ys}	yield strength of hoops steel.
P_c	experimental ultimate axial load of plain concrete column.
P_t	experimental ultimate axial load.
P_u	ultimate design axial load of reinforced concrete column.
P_w	allowable design axial load of reinforced concrete column.
S_b	bolts pitch.
s	distance between hoops.
T	Tension force in 2 bolts due to tightening.
T_b	Total force in bolt
T_y	proof load of bolt
T1	Tension force induced in 2 bolts when axial load is applied.
t_j	Thickness of jacket shell.
α	factor.
ρ	percentage of lateral reinforcing steel volume.
ΔP	percentage of ultimate axial load increment.
σ	radial lateral stress due to initial applied load.
σ_a	active lateral radial pressure.
σ_j	radial stress induced in jacket shell.
σ_{jy}	yield stress of shell material.
σ_p	passive lateral radial pressure.
σ_s	passive lateral radial pressure due to hoops interaction.

REFERENCES

1. U. A.,Ersoy, T. Tankut, and R., Suleiman, "Behavior of Jacketed Columns", ACI Structural Journal, Vol. 90, No. 3, pp. 288-293, (1993).
2. M. Rodriguez, and R. Park, "Seismic Load Tests on Reinforced Concrete Columns Strengthened by Jacketing", ACI Structural Journal, Vol. 91, No. 2, pp. 150-159, (1994).
3. H., Saadatmanesh, M.R. Ehsani and M. W. Li, "Strength and Ductility of Concrete Columns Externally Reinforced with Fiber Composite Straps.", ACI Structural Journal, Vol. 91, No. 4, pp. 434-447, (1994).
4. H. Saadatmanesh, M.R. Ehsani, and L. Jin, " Seismic Strengthening of Circular Bridge Pier Models with Fiber Composites.", ACI Structural Journal, Vol. 93, No. 6, pp. 639-647, (1996).
5. Y. Xiao, and Ma Rui, " Seismic Retrofit of RC Circular Columns Using Prefabricated Composite Jacketing.", J. Struct. Div., Proc. ASCE, Vol. 123, No. 10, pp. 1357-1364, (1997).
6. Y.H. Chai, M.J.N. Priestley, and F. Seible," Seismic Retrofit of Circular Bridge Columns for Enhanced Flexural Performance", ACI Structural Journal, Vol. 88, No. 5, pp. 572-584, (1991).
7. M. J. N. Priestley, F. Seible, Y. Xioa, R. and Verma, "Steel Jacket Retrofitting of Reinforced Concrete Bridge Columns for Enhanced Shear Strength-Part 1: Theoretical Considerations and Test

Design", ACI Structural Journal, Vol. 91, No. 4, pp. 394-405, (1994).

8. M.J.N. Priestley, F. Seible, Y. , Xioa, R. and Verma, "Steel Jacket Retrofitting of Reinforced Concrete Bridge Columns for Enhanced Shear Strength-Part 2: Test Results and Comparison with Theory", ACI Structural Journal, Vol. 91, No. 5, pp. 537-551, (1994).

9. Y. Xiao, M.J.N. Priestley and F. Seible, "Seismic Assessment and Retrofit of Bridge Column Footings.", ACI Structural Journal Vol. 93, No. 1, pp. 79-94, (1996).

10. 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).

11. International Standard ISO 898-1, Mechanical properties of fasteners - Part 1: Bolts, Screws and Studs, (1988).

12. S.R. Razvi and M. Saatcioglu, "Confinement of Reinforced Concrete Columns with Welded Weir Fabric", ACI Structural Journal, Vol. 86, No. 5, pp. 615-623, (1989).

13. S.A. Sheikh and M.T. Toklucu, "Reinforced Concrete Columns Confined by Circular Spirals and Hoops", ACI Structural Journal, Vol. 90, No. 5, pp. 542-553, (1993).

14. R.W., Furlong, G.L. Fenves and E.P. Kasi, "Welded Structural Weir Reinforced for Columns" , ACI Structural Journal, Vol. 88, No. 5, pp. 585-591, (1991).

15. D. Cusson, P. and Paulter, "High-Strength Concrete Columns Confined by Rectangular Ties", J. Struct. Div., Proc. ASCE, Vol. 120, No. 3, pp. 783-804, (1992).

16. ACI committee 318, "Building Code Requirements for Reinforced Concrete and Commentary (ACI 318-89/ACI 318R-89)", American Concrete Institute, Detroit, (1989).

17. Research Center for Housing, Building and Physical Planning, Egyptian Code for the Design and Construction of Reinforced Concrete Structures, Cairo, Egypt, (1989).

18. K.J. William, E.P. and Warnke, "Constitutive Model for Triaxial Behavior of Concrete", Proc., International Association for Bridge and Structural Engineering, Vol. 19, pp. 1-30, (1975).

19. J.B. Mander, M.J.N. Priestley and R. Park, "Theoretical Stress-Strain Model for Confined Concrete", J. Struct. Div., Proc. ASCE, Vol. 114, No. 8, pp. 1804-1825, (1988).

20. F.E. Richart, A. Brandtzaeg and R.L. Brown, "A Study of the Failure of Concrete Under Combined Compressive Stresses", Bulletin 185, Engineering Experimental Station, Univ. of Illinois, Champaign, (1928).

21. J. Gardner, R. and Jacobson, "Structural Behavior of Concrete Filled Steel Tubes.", ACI Journal, No. 64-38, pp. 404-414, (1967).

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قميص حديدى جديد لتقوية الأعمدة القصيرة من الخرسانة المسلحة ذات القطاع الدائرى والمعرضة لقوى ضغط محورية

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ملخص البحث

يقترح البحث أستخدام قميص حديدى جديد لتقوية الأعمدة من الخرسانة المسلحة ذات القطاع الدائرى. تركيب القميص يوفر الوقت و المجهود وكذلك التكاليف بالمقارنة الى الوسائل المتاحة حاليا. طريقة تركيب القميص تولد أجهاد ضغط عمودى على كامل محيط العمود . العمود المقوى بهذا القميص يصبح بة مميزات الأعمدة الحديدية ذات المقطع المفرغ والمملوءة بالخرسانة. القميص يمكن أستخدامة لأصلاح و تقوية الأعمدة الخرسانية الموجودة فعلا وكذلك فى الأعمال الجديدة بصرف النظر عن حالة العمود أذا كان محملا أم لا. تم أختبار ١٦ عمود قصير ذوى قطاع دائرى بتعريضهم لقوى ضغط محورى حتى الأنهيار. مقاومة الخرسانة للضغط المحورى زادت أكثر من الضعف وكذلك تم أقتراح طريقة لتصميم القميص للحالات المختلفة.