قميص حديد جديد لتقوية الأعمدة القصيرة من الخرسانة المسلحة

و ذات قطاع دائرى

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ملخص البحث

يقترح البحث أستخدام قميص حديد جديد لتقوية الأعمدة من الخرسانة المسلحة ذات القطـــاع الدائرى. تركيب القميص يوفر الوقت و المجهود وكذلك التكاليف بالمقارنـــة الـــى الوســـائل المتاحة حاليا. طريقة تركيب القميص تولد أجهاد ضغط عمودى على كامل محيط العمـــود . العمود المقوى بهذا القميص يصبح بة مميزات الأعمـــدة الحديـــد ذات المقطـــع المفـــرغ و المملوءة بالخرسانة. القميص يمكن أستخدامة لأصلاح و تقوية الأعمدة الخرسانية الموجـــودة فعلا و كذلك فى الأعمال الجديدة بصرف النظر عن حالة العمود أذا كان محمـــلا أم لا. تـــم أختبار ١٧ عمود قصير ذا قطـــاع دائـــرى بتعريضـــهم لقـــوى ضـــغط محـــورى حتـــى الأنهيار.مقاومة الخرسانة للضغط المحورى ذادت أكثر من الضعف. النتائج المستخرجة تـــم عرضها فى الجداول ٣ و ٤. تم أقتراح طريقة لتصميم القميص للحالات المختلفة.

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New steel jacket for retrofitting of 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 the repair and strength of already existing columns and for new work regardless of column loading state; i.e. loaded or unloaded. A total number of 17 cylindrical short columns were tested under concentric axial loads until failure. Concrete axial strength was increased in value more than twice. The results obtained are presented in tables 3 and 4. Design method.

Key words:

jacket-steel, column-concrete, column-Circular, column-short , strength-axial, strength-confined, strength-concrete.

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Introduction

The earthquake of October 1992 in Egypt affected many of the reinforced concrete multistory buildings. Some are subjected to sever damage and became unable to sustain their service loads. Others suffered from cracks appeared in the different structural elements. This refers to one or all of the following reasons. The majority of buildings have been built without adequate resistance to earthquake forces while others were not build according to the codes or even engineered at all. The absence of quality control tests on building materials specially in the 70's is another reason. Retrofitting of structures after damage caused by earthquake and/or to comply with new recent codes to improve their resistance against earthquakes without radical alterations in their appearance became a need. Deficiencies often found in columns and beams-columns joints. For seismic design, beams are seen to be the element best suited to tolerate inelastic deformations without failure of the structure. To discourage plastic hinging in columns, most building codes have adopted the design concept of " strong column and weaker beam". Retrofitting of reinforced concrete column depends on jacketing the column either for strengthening or repairing. Several techniques are presented in the literature for columns jacketing. They depends on the material used for the jacket.

Jacketing of column using additional reinforced concrete is commonly used in Egypt and abroad. Experimental studies (Erase et al., 1993 and Rodriguez ; 1994) showed that repairing and strengthening jackets, made after unloading the column under consideration, improved strength and ductility. Erase et al. (1993) included in their study the case when repair jackets are made for loaded columns. The columns were tested under concentric axial loads. The results indicated that "the jacketing was not very successful and the column could carry only 50% of the axial load carried by a similar monolithic specimen." 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. Saadatmanesh et al. (1994) proposed a technique for strengthening columns using high strength composite straps. The straps are wrapped around the column in a continuous spiral and/or discontinuous rings. The ends of the straps are either mechanically coupled or epoxy bonded to the column. The results obtained indicate improvement of the confinement and hence in the strength and ductility. In another study by Saadatmanesh et al. (1996), passive retrofit scheme was implemented. Unidirectional straps of E-glass were wrapped in multiple layers around the column to form external hoops. In one case, active retrofit scheme was carried out. The composite strap were slightly oversized for the column and the resulting gap was initially injected with pressurized epoxy resin. Initial tension stresses are expected to induce in the straps. Active pressure is created around the column. Both the passive and active systems provided additional confinement to existing concrete core. "The improvements resulting from the active retrofit scheme, as compared to the passive scheme do not seem to justify the additional cost associated with the active retrofit scheme." Xiao and Ma (1997) investigated Another system for retrofitting reinforced concrete columns with lap spliced longitudinal steel. The retrofit system 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 been proved to be an effective measure to retrofit bridge columns for increased strength and ductility (Chai et al. 1991; Priestley et al. 1994; Xiao 1996). Circular cylindrical steel jackets are constructed in two half-shells slightly oversized for easy installation. The two half's 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. Experimental and analytical studies have been carried out to investigate the behavior of this type of columns considering loading capacity (Salani 1964; Gardner 1967; Knowles 1969 and Bode 1976), the effect of eccentric loading (Neogi 1969 and Rangan 1992) , rectangular cross sections (Shakir-Khalil 1989 &1990 and Hanbin Ge 1992), the bond strength between the concrete and steel (Hunaiti 1991 and Shakir-Khalil 1993). Design procedures are proposed by Bradford (1996) and Wang et al. (1997). Further review of other studies about the behavior of concrete - filled tubular steel columns is presented by Shams et al. (1997). This type of composite 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. The confinement created by the steel casing enhances the concrete strength. The concrete would be under a triaxial state of stress and would prevent the inward buckling of the steel section. The system 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 strength of already existing columns and for new work regardless of column loading state; i.e. loaded or unloaded. A total number of 17 cylindrical short columns were tested under concentric axial loads until failure. the results obtained are presented in tables 3 and 4. Design method are proposed.

Research Significance

The results obtained are used to measure the enhancement gained due to the use of the proposed jacket considering 1) concrete axial strength and 2) the loading carrying capacity of retrofitted columns. The use of the proposed steel jacket is validated and design procedure is proposed.

The proposed Steel jacket

The jacket is cylindrical and made in two half-shells of hot rolled steel sheet. It would be installed around the column as external continuos 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. Further, this would minimize any flexural enhancement which might cause excessive forces to develop in adjacent members, Priestley et al. (1994). The ends of the two shells at one side are prepared for welding in 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 become 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 forming 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 hence 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, figure 1-b.

Experimental program

The behavior of reinforced concrete columns retrofitted using the proposed steel jacket and subjected to concentric loads are studied experimentally in this paper. A total number of 17 columns were tested under concentric axial load. The dimensions and details of the specimens are presented in table 1.The variables considered are 1) concrete strength, 2) longitudinal steel ratio, 3) degree of lateral pressure induced on the concrete column and 4) loading state of the column. To verify the use of the proposed steel jacket for 1) new work and 2) strengthening and 3) 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 JNRA25^{s} AND JORA25^{s} were loaded only to 80% of their ultimate loads and the tests were stopped. The specimens were investigated visually. Specimen JNRA25^{r} was loaded to 68% of its ultimate capacity. The test was stopped, the steel jacket was removed and the concrete column was reloaded until failure. 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 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 until failure. The bolts were then tightened and the retrofitted column was loaded again until failure. The comparisons of the results of the specimens of group A, with respect to particular variable, to those of the other groups would indicate the confinement efficiency and effect provided by the proposed steel jacket.

Test Specimen

i - Steel Jacket

It was seen, for this study, that their is no need to manufacture the steel jacket in two half shells. The concrete column is short and not connected at its ends to other structural elements. A one piece jacket would cause no problem in the installation procedure and not affect the obtained results. A number of 11 steel cylinder jackets having a length of 600 mm was 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 40 X 40 X 4 , 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. The angles were adjusted so that the centers of the bolts holes become coincide and distance of 20 mm is left between their standing legs. 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. The welding was carried out by manual metal arc welding process. The electrode was of diameter 3.25 mm, length 350 mm and class E4332 R complying with DIN 1913.

ii - Concrete column

A total number of 17 concrete columns of 600 mm length and nominal diameter of 190 mm were casted. The reinforcing details are presented in table1. Hoops were concentrated at the top and bottom regions of the column to prevent premature failure and distributed as shown in figure 3-a. In only one specimen OSA, prepared for comparison purposes, hoops were used as shown in figure 3-b. The steel jackets were used as forms. The columns were casted vertically. After 24 hours, the forms were removed and the concrete were cured for 28 days in 100% relative humidity.

Material properties

i - Concrete

Two concrete mixes designated as A and B were designed with specified 28 days strength of 20.42 and 18.13 N/mm^2 respectively. These values are in the normal range of concrete strength used in the 70's. For each mix, six 150 X 300 mm concrete cylinders were tested under axial load. The strength values were calculated and their average values were used. Mix A were used in 14 columns while B in 3 columns.

ii - Reinforcement

Deformed steel bars of 12 and 16 mm diameter were used for longitudinal reinforcing. The length of the 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. A concrete cover of 20 mm was provided for the reinforcement at the circumference of the column. 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.

iii - 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.

iv - Bolts

Bolts of size M 10 and grade 8.8 complying with DIN 931 were used for tightening the jacket. This size was used for the following reason. The standing legs of the welded angles became not parallel to each other. More clearance is required for inserting the bolts in the coinciding holes in the two angles. This necessitated the use of bolts size M 10 in holes of 13 mm diameter. Two steel strips of 10 mm thickness, 25 mm width, 590 mm length and grade 37 were tapered. They were placed under both bolts heads and nuts as washers to compensate for the unparallel surfaces to which the bolts heads and nuts react. Nuts of grade 8 complying with DIN 934 were used. The minimum yield and ultimate strength specified in DIN 18800 for this grade of bolts are 640 N/mm^2 and 800 N/mm^2 respectively. The minimum ultimate tensile load and proof load for this size and grade of bolt specified in ISO 898 - 1 (1988) are 46400 N and 33700 N respectively.

Specimen Assembly

Concrete columns, tested without steel jacket, were externally confined at the top and bottom regions by steel collars. This prevented premature failure of the column at these regions, (Salim et al. 1989 and shamim et al. 1993). For retrofitted columns, the steel jacket was installed and the bolts were tightened to a defined value using torque wrench, 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 and hence losses in the induced forces. The tightening procedure was repeated again 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 digits were used to designate concrete columns. The first two digits are PC for plain concrete, NR for normal longitudinal reinforcing, OR for over longitudinal reinforcing or OS for over lateral reinforcing. The third digit 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 ft - lb 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 made in the steel jacket specially for this purpose at 85 mm above and below the mid height of each column, 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 able us to recorded the descending part of the load deformation relationship. However, the technique is seen to be suitable for the jacketing procedure utilized in this study and was used before by Furlong et al. (1991) and Cusson et al. (1994).

Test setup and procedure

Universal testing machine of 3000 KN loading capacity was used for testing the columns. The specimen was placed in the center of the testing machine,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. When the concrete cover spills, the loading procedure continued until significant drop in load was recorded. The maximum load recorded is considered the ultimate load of the column.

Observed Behavior and Test Results

i- Concrete Columns

No steel jackets were used for these columns, group A of table 1. Concrete columns were loaded until failure. The ultimate loads are presented in table3. Figures 5 to 7 show the failure modes at the end of the tests. Columns PCA and PCB were unreinforced. They were tested to obtain the unconfined concrete axial strength of columns having the same size as the confined columns avoiding scale effect. The failure of PCA was characterized by concrete crushing at mid height of column, figure 5-a. PCB failed in a diagonal plane as shown in figure 5-b. The reinforced 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 were destroyed and the longitudinal steel bars had buckled, figures 6 and 7. Table 3 includes computed and measured values determined as follows:

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

$$P_o = \alpha\, f_c\ A_c + A_s\ F_y \qquad 2$$

$$P_u = 0.35\ F_{cu}\ A_c + 0.67\ A_s\ F_y \qquad 3$$

$$P_w = f_{ca}\ A_c + 0.44\ A_s\ F_y \qquad 4$$

The enhancement in column loading capacity due to the use of longitudinal reinforcement is indicated by the percentage ΔP %, table 3. P_c is the experimental ultimate load obtained of unreinforced concrete column having the same concrete grade of the column considered. Equation 2 is of the ACI 318 Committee. The value of α was made equal to 0.85 and 1 in computing the values in col. 4 and 5 of table 3 respectively. Equations 3 and 4 are of the Egyptian Code for the design of reinforced concrete structures, (1996). Equation 3 is for ultimate strength limit state while equation 4 is for working stress method. The calculated loads are compared to the experimental values.

ii - Jacketed Columns

Steel jackets were used for the concrete columns, specimens 7 to 17 of table1. Axial concentric load was applied to the concrete column and not to the steel jacket. The ultimate loads obtained are presented in table 4. The result of each column is compared to that of column having the same designation but not jacketed, table 3. Figures 8 and 9 show the failure modes of the concrete column and the steel jacket. Figure 10 shows the fracture of JPCA10. The result of columns JNRA25^s and JORA25^s visual inspection are presented in figures 11-a and 11-b respectively. Figures 12 and 13 compares between the relationships obtained of the applied loads and axial strains of different columns, retrofitted and not. Table 4 includes computed and measured values determined as follows:

$$P = P_t - A_s\ F_y \qquad 5$$

$$f_{cc} = P / (A_c - A_s) \qquad 6$$

$f_{cc} = f_c\ (-1.254 + 2.254\ \ [1+(7.94\ \ f_L /f_c)]^{1/2}\ \ - 2\ f_L / f_c)$ 7

$R\ = (f_{cc}-f_c)/f_L$ 8

Retrofitted columns of groups B and C were failed in similar manner. At failure, drop in the applied load was recorded. Part of the concrete cover to which the plate forms of the displacement gauges were attached were spilled off. Local buckling occurred at the sides of the 40 X 40 openings made in the steel jacket, figures 8. When removing the steel jacket, vertical cracks were observed in the concrete column in the length direction. This is concentrated at the region of the jacket connection; i.e. the over lapped sides of the jacket; figures 9. No concrete crushing was observed. Concrete cover did not spell off except at the square openings. The column was still in one unit but the concrete become fragile. The concrete column of JPCA10 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 JNRA25^s and JORA25^s respectively. They were loaded to 1300 and 1669 KN respectively. Most of the cracks were found in the side of the jacket connection rather than in the other side. Column JNRA25^r was loaded to 1100 KN. The steel jacket was removed and the column was then loaded until failure, table 4. Imposing relatively high compressive stress level to the concrete column when its jacketed did not effect the ultimate load of the column when the jacket was removed.

The columns of group C were tested to study the case of strengthening already existing loaded columns. Axial load was applied to the concrete columns. When the applied load reached 600 KN the bolts were tightened to a torque of 25 lb - ft. The loading procedure was then continued until failure. The ultimate load of JNRA25-L600 is slightly greater than those of JNRA25 and JNRA10. The failure mode observed is typical to that of the columns in group B. The columns of group D were tested to study the case of repairing already failed concrete columns. The failure of NRA* is characterized by vertical cracks started at the column top and continues through the column length at the connection region. It worth mentioning that the over lapped sides of the connection was totally cut till the edges of the angles in this particular steel jacket. The bolts were then tightened and the retrofitted column was loaded again. At load 1375 KN, sounds were heard. The test was stopped at load 1450 for safety. The concrete column PCA* of specimen 17 was fractured during handling for testing. The fractured parts of the column were collected together, put in a steel jacket and subjected to axial load. At load 490 KN, the load dropped significantly. The bolts of the jacket were then tightened to torque of 25 lb - ft and the column was loaded again. The retrofitted column was failed at load equal to that of JPCA10 of the first group.

Analysis of Test Data

i - Concrete Columns, table 3

The results of NRA and NRB agree with equation 2, col 4. The concrete contribution is estimated by the term 0.85 f_c A_c. However, unreinforced columns showed higher ultimate strength even when α of equation 2 is made equal to 1, col 5. When using F_{cu} instead of f_c and making $\alpha = 1$, the calculated loads would equal 1.04 and 1.05 of the experimental values obtained of PCA and PCB respectively. The increase in the ultimate loads of columns NRA and NRB in comparison to unreinforced concrete columns is found to be 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. This indicates that, in this particular case, equation 2 under estimates and over estimates the concrete and steel contributions respectively. This would refer 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, column ORA. Equation 2 over estimated the ultimate load in this case, col 4 and 5. The values obtained using equations 3 and 4 for this case are not consistent with those obtained for columns NRA and NRB. In contrast, increasing the volumetric ratio of lateral confining steel ρ =1.64% and reducing the ratio s/d_s to .26 as column OSA, displayed large load enhancement. This is due to the satisfactory performance of circular hoops at these lateral reinforcing conditions (Shamim et al. 1993). Equations 2, 3 and 4 under estimated column OSA loading capacity in comparison to NRA and NRB. The discrepancy in the values obtained using equations 2, 3 and 4 refers to the following reason. The equations 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. Part of the answer is related to the existence of quality control tests and engineering supervision required in codes.

ii - Jacketed Columns , table 4

Equations 5 and 6 were used to calculate the experimental values of concrete axial strength in jacketed columns. Equation 7 was used to calculate the corresponding lateral radial pressure causing confinement. This equation was described by William and Warnake (1975) and adopted by Mander et al. (1988) in his theoretical model for confined concrete. Concrete axial strength was enhancement more than twice, col 4. This caused increase in the ultimate axial load of jacketed columns in comparison to their counter parts of table 3 as shown in col 3. Tightening of the bolts in the jacket would induce active lateral radial pressure σ_a, equation 9 and figure 14-a. The confining action of the jacket in addition to the hoops, if exists, would induce passive lateral radial pressure, σ_p and σ_s respectively, when axial load is applied. Equations 10 to 12 and figures 14-b and 14-c illustrates the relationships between the different components of induced radial pressure at failure.

$$\sigma_a \; S \; d/2 = T/2 \qquad 9$$

$$2 \; F_{ys} \; A_{ss} = \sigma_s \; s \; d_s \qquad 10$$

$$f_L = \sigma_s \; k + \sigma_a + \sigma_p \qquad 11$$

$$T1/2 = S \, (\sigma_a + \sigma_p) \, d/2 \qquad 12$$

The value of f_L is the actual affecting radial pressure while σ_p is the net value applying on the concrete column. When the value of factor k in equation 12 is taken equal to 1.4, the summation of σ_a and σ_p of column JNRA10 would equal to that of JPCA10. The values of the ratio R , col 5, obtained using equations 5 to 8 are in the range found by Richart et. al (1928). This applies to all the columns except that of JORA25-L600. 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 col 6 and 7.

$$2 \text{ force in bolt } = 2\, \sigma_j\; t_j \quad S = T + T1 \qquad 13$$

Increasing the tightening degree of the bolts, as the case of JNRA25 in comparison to JNRA10, 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, equation 12.

Failure of jacketed column would initiate due to yielding of the bolts and/or the shell. The occurred elongation would reduce the confining lateral pressure on the concrete and hence its axial strength. The concrete column would be cracked as described above. Bolts were subjected to loads higher than their proof loads at failure, col 7. This would cause elongation in the bolt and drop in the applied axial load. If the yielding was occurred first in the shell, relatively large radial strain is expected to occur and hence significant reduction in applied load. The change in concrete axial strength as the case of JNRB10 did not affect the ratios f_{cc} / f_c and X values 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 9 would be modified as follows:

$$(\sigma_a + \sigma)\, S\; d/2 = T/2 \qquad 14$$

where σ is the lateral radial pressure produced due to the initial applied axial load. This would reduce the active pressure 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 JNRA25-L600. Column JORA25-L600 showed relatively high ultimate axial load and strength. The authors find no explanation to these results at the time being.The retrofitting of the concrete columns of specimens 16 and 17, although the concrete was already fractured, gave nearly the same results of their counter parts of group B. In fact, this result agrees with the findings of Gardener et. al. (1967), specimens 31 and 32.

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. This degree depends on the concrete column size and its circumference regularity. The data required for the design procedure are 1) the required enhancement of the column loading capacity, 2) strength of standard concrete cylinder, 3) column geometrical cross section details and 4) the longitudinal and lateral reinforcing steel details. The value of f_L is calculated using equations 5 to 7 where 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 9 to 12. The shell thickness and the bolts size, grade and pitch would be defined according to the value obtained from equation 13.

In this study, the value of bolt proof load and equations 5 to 13 were used to obtain the value of f_L and the design ultimate load of retrofitted columns. the results are compared to the experimental values obtained, col 8. The calculated values are in acceptable range of the experimental results. This does not agree with the results of specimens 14 and 15 due to neglecting the radial stress σ as explained above.

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 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 enhancement more than twice, col 4 of table 4. This caused increase in the ultimate axial load of jacketed columns in comparison to their counter parts of table 3. Increasing the tightening degree of the bolts would reduce the net value of applying lateral radial pressure on the concrete and increasing 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 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 as the case of group C columns. The jacket showed its success in retrofitting of already failed columns as the case of group D.

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APPENDIX II - Notations

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_{cu}	strength of standard concrete prism of 15x15x15 cms at 28 days.
F_y	yield strength of reinforcing steel.
F_{ys}	yield strength of hoops steel.
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.
k	factor.
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	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.

1 - Jacket shell 2 - Welded angles 3 - Bolts and nuts 4 - tapered washers
2 - Welded angles 4 - Tapered washers

Figure 1 : Details of the proposed steel jacket cross section.

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

(a) Cross section (b) Elevation

Figure 3 : Details of reinforcing steel in concrete columns.

(a) Normal lateral reinforcing (b) Over lateral reinforcing - OSA

Figure 4 : Specimen in testing machine.

Figure 5 : Failure of plain concrete columns.

(a) column PCA (b) column PCB

Figure 6 : Failure of reinforced concrete columns, NRA and NRB.

(a) column NRA (b) column NRB

Figure 7 : Failure of reinforced concrete columns, ORA and OSA.

(a) column ORA (b) column OSA

Figure 8 : Column JNRB10 at failure.

Figure 9 : Column JORA25-L600 at failure.

Figure 10 : Fracture of column JPCA10.

Figure 11 : Observed cracks at 80% of ultimate loading capacity.

(a) column $JNRA25^s$ (b) column $JORA25^s$

Figure 12 : Applied load - axial strain relationship of columns NRA, $JNRA25^s$, $JNRA25^r$ and JNRA25-L600.

Figure 13 : Applied load - axial strain relationship of columns ORA, $JORA25^s$, PCA and JORA25-L600.

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