Materials and Structures (2014) 47:1715–1728
DOI 10.1617/s11527-013-0146-x

ORIGINAL ARTICLE

RC columns externally strengthened with RC jackets

G. Campione · M. Fossetti · C. Giacchino · G. Minafò

Received: 2 October 2012 / Accepted: 9 July 2013 / Published online: 19 July 2013

Abstract In this paper the behaviour in compression of RC columns externally strengthened with concrete jacketing is analysed and a cross-section analysis of the jacketed member under axial load and bending moment is developed. The focus was to study the effect of confinement of concrete jacket on concrete core and the behaviour of compressed bars with buckling effects. Some other important aspects such as shrinkage, creep, old to new concrete surfaces and bond split effects were not included in the model because: the use of thick non-shrink grout jacket and a well-roughened surface of old-to-new concrete was supposed; long term effects were included though corrective coefficients for monolithic behaviour proposed in the literature. A preliminary validation of the model adopted was made referring to the short-term experimental data available in the literature on the compressive behaviour of RC columns strengthened with concrete jacketing. Good agreement was obtained with the available data in terms of moment-axial force domains and moment–curvature diagrams. The analysis showed the effectiveness of this reinforcing technique in improving both the strength and the ductility of RC cross-sections of columns, highlighting the importance of an accurate choice of strengthened materials (concrete grade, thickness of jacket, space and diameter of stirrups, etc.).

Keywords Concrete columns · Concrete jacketing · Confinement · Moment–curvature diagram

List of symbols

a_{sc}	Stiffness of the internal stirrup
a_{sj}	Stiffness of the external stirrup
b	Side of square cross-section
B	Side of the external jacket
c_c	Concrete cover in the core
c_j	Concrete cover in the jacket
d	Thickness of the external jacket
d_{bc}	Diameter of longitudinal bar in the core
d_{bj}	Diameter of longitudinal bar in the jacket
e	Eccentricity of the axial load with respect to the column axis
E_c	Elastic modulus of concrete
E_h	Hardening modulus of steel
E_r	Reduced modulus of steel proposed by Papia and Russo for buckling effects
E_s	Elastic modulus of steel
E_{sec}	Secant modulus of concrete
f_{c0}	Compressive strength of unconfined concrete
f_{cc}	Compressive strength of confined concrete
f_{cj}	Compressive strength of concrete in the jacket
f_{ct}	Tensile strength of concrete

G. Campione (✉)
DICAM, University of Palermo, Viale delle Scienze, 90128 Palermo, Italy
e-mail: studioingcampione@libero.it

M. Fossetti · C. Giacchino · G. Minafò
Facoltà di Ingegneria e Architettura, University of Enna "Kore", Cittadella Universitaria, 94100 Enna, Italy

f_l	Maximum equivalent confinement pressure
f_{lc}	Confinement pressure due to the core reinforcement
f_{lj}	Confinement pressure due to the external jacket
f_y	Yield stress
f_{yc}	Yield stress of longitudinal bar in the core
f_{yj}	Yield stress of longitudinal bar in the jacket
f_{ysc}	Yield stress of stirrups in the core
f_{ysj}	Yield stress of stirrups in the jacket
k	Stirrup stiffness for unit length
k_{pc}	In-plane effectiveness coefficient of confinement pressure induced by internal stirrups
k_{vc}	In-elevation effectiveness coefficient of confinement pressure induced by internal stirrups
k_{ej}	In-plane effectiveness coefficient of confinement pressure induced by external stirrups
k_{vj}	In-elevation effectiveness coefficient of confinement pressure induced by external stirrups
M	Bending moment in the column
N	Axial load in the column
s_j	Pitch of the stirrups in the jacket
s_c	Pitch of stirrups in the core
ε	Axial strain
ε_{c0}	Axial strain corresponding to the peak stress in unconfined concrete
ε_{cc}	Axial strain corresponding to the peak stress in confined concrete
ε_y	Yield strain
ϕ_{sc}	Diameter of stirrups in the core
ϕ_{sj}	Diameter of stirrups in the jacket
σ_c	Compressive stress in the concrete
ω_{st}	Mechanical ratio of the transverse stirrups

1 Introduction

Nowadays concrete jacketing is one of the most common techniques used to increase the bearing capacity and ductility of existing reinforced concrete (RC) columns [1, 9, 11–15, 18, 22, 24], [3–5, 23]. It is done by casting a new RC layer ("jacket") around the original column, in order to enlarge the cross-section and also inducing confining action in the inner core. Concrete jacketing can be made in two different manners; if the external shell and the original column are equally high, the jacket is directly loaded (Fig. 1a), while if the new concrete layer is shorter than the column (see Fig. 1b), a gap is interposed between the external shell and beams placed on the top and bottom of the column. In the latter case the concrete jacket is indirectly loaded.

The concrete jacket induces a confinement effect on the inner column and the lateral confining pressure along the side of the column does not have uniform distribution, but reaches a higher value in the corners and has a minimum in the mid-side. These confinement pressures produce increases in the loading carrying capacity and reduce the risk of buckling of longitudinal bars.

The effect of old-new concrete interface, as observed in Vandoros and Dritos [24] and Julio et al. [13], plays a lower role with respect to the long time effect including shrinkage effect. If concrete surface of old concrete is not roughened, the reduction in the effectiveness of composite columns, in terms of flexural capacity, is almost 10 %. If interfaces were well roughened, these effects are negligible [1, 13]. Therefore, the model was developed by neglecting these effects because the interface of old concrete was supposed well roughening before jacketing.

The effect of steel bar-concrete slippage reflects mainly in an additional rotation at the base of the compressed members (fixed end rotation) and mainly reflects under cyclic loading. Under monotonic loading, a reduction of flexural capacity is expected. In the current paper these effects were not considered. Further studies will be addressed to include these effects that mainly reflect on the behavior of structural members (cantilever, columns) under flexure and axial force.

The confinement effect in concrete jacket is negligible especially for reduced thickness and because of the by-axial state of stresses (compression-tension). This is also confirmed by Altun [1], which noted that the mechanical behavior of jacketed RC beams is similar to those of ordinary RC beams of the same dimensions.

Another important aspect which has to be considered when designing a concrete jacket is the past loading history of the column [3]. If the column is strengthened under loading, the full load-carrying capacity of the retrofitted member cannot be reached, considering the external layer coupled to the inner core. In order to take these effects into account, one of the theoretical approaches available in the literature [3] proposes reducing the axial capacity of the strengthened column.

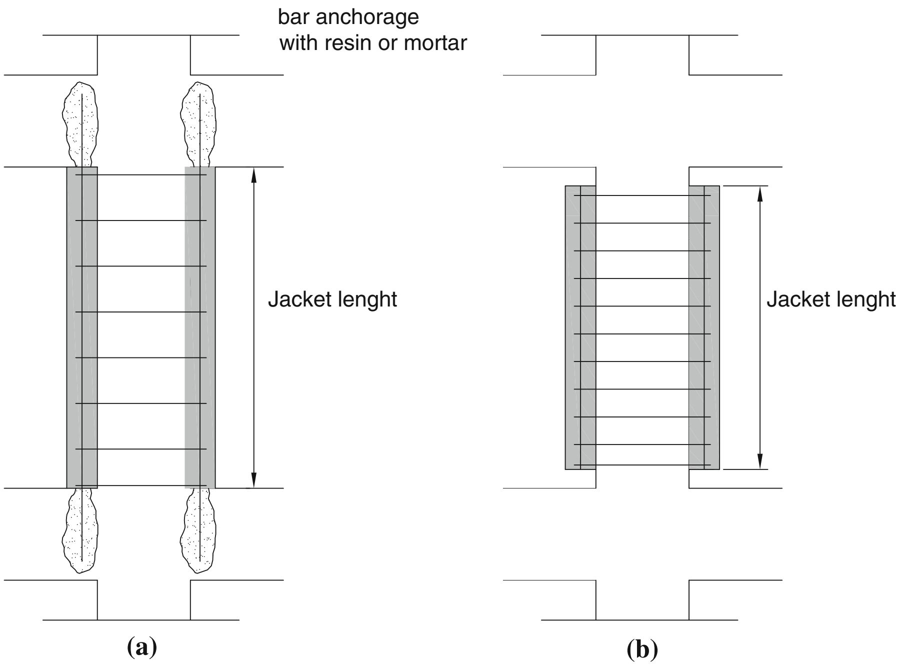


Fig. 1 Concrete jacketing techniques; **a** jacket directly loaded; **b** jacket indirectly loaded

From practical point of view in the past few years, some studies have proposed design rules for concrete jacketing techniques [18]; specifically, these can be summarized as follow: the strength of the new materials utilised for the jacket must be greater than that of the column; the thickness of the jacket should be at least 4 cm for shotcrete application and 10 cm for cast-in situ concrete; the reinforcement should be not less than four bars for four-side jacketing and minimum bar diameter 14 mm; the ties should be minimum 8 mm and at least 1/3 of the vertical bar diameter; the vertical spacing is at most 200 mm and close to the joint must not exceed 100 mm. In addition, the spacing of the ties should not exceed the thickness of the jackets. Further, the surface should be moistened before placing shotcrete and the existing concrete must be heavily sandblasted and cleaned of all loose materials, dust and grease obtaining in this way a well-roughened surfaces.

2 Theoretical model for strengthened columns

The focus of the proposed model was the study of the effects of confinement induced by external and internal stirrups on concrete core and the behaviour of longitudinal bars in tension and in compression including second order effects. It has to be noted that in general confinement provides beneficial effects in compression rather than in flexure; however different works focused about the compressive behaviour of RC jacketed columns [3, 11, 21], while in the contrary a few of works highlighted the effect of RC jackets when the column is subjected to axial load and bending moment. The current work aims to evaluate the efficiency of the RC jacketing technique also in the case of combined effect of axial load and bending moment, taking into account confinement effects on concrete and local buckling of longitudinal bars.

The effect of shrinkage, old to new concrete interfaces and slippage of longitudinal bars are not considered for the reasons mentioned in the previous section. It is here stressed again that some of these effect, are negligible while some others are important. To take into account of these effects refined 3D finite element analyses are required.

The case examined here is that of a concrete member with a square cross-section with side b (see Fig. 2) externally strengthened with a concrete jacket of external side B and thickness δ. The jacket was reinforced with longitudinal bars of diameter d_{bj} and close stirrups of diameter ϕ_{sj} at pitch s_j. f_{yj} is the yield stress of the longitudinal steel in the jacket and f_{ysj} is the yield stress of the steel stirrups. The presence of pre-existing longitudinal bars of diameter d_{bc} and transverse stirrups of diameter ϕ_{sc} placed at pitch s_{sc} was also considered. f_{yc} is the yield stress of the longitudinal bars in the core and f_{ysc} is the yield stress of the stirrups. The columns were subjected to the coupled effects of axial load N and bending moment M, giving eccentricity $e = M/N$. Cases of directly and indirectly loaded concrete jackets (Fig. 1 a, b), also including second-order effects for longitudinal bars, were considered.

3 Modelling of concrete behaviour

The concrete model adopted here was the well-known model of Mander et al. [16] leading to stress–strain curves for effectively confined and unconfined concrete. It is based on the following relationship:

$$\sigma_c = \frac{\frac{\varepsilon}{\varepsilon_{cc}} \cdot f_{cc} \cdot r}{r - 1 + \left(\frac{\varepsilon}{\varepsilon_{cc}}\right)^r} \tag{1}$$

With

$$r = \frac{E_c}{E_c - E_{sec}}, \tag{2}$$

where $E_c = 5000 \cdot \sqrt{f_{co}}$ in MPa and $E_{sec} = \frac{f_{cc}}{\varepsilon_{cc}}$,

With f_{c0} and ε_{c0} the strength and the strain of unconfined concrete, f_l the effective confinement pressure, while f_{cc} and ε_{cc} are the strength and the strain of confined concrete evaluated according to Mander et al. [16] as:

$$f_{cc} = f_{c0}\left[2.254\sqrt{1 + \frac{7.94 \cdot f_l}{f_{c0}}} - 2 \cdot \frac{f_l}{f_{c0}} - 1.254\right] \tag{3}$$

$$\varepsilon_{cc} = \varepsilon_{co} \cdot \left[1 + 5 \cdot \left(\frac{f_{cc}}{f_{c0}} - 1\right)\right] \tag{4}$$

To determine the maximum confinement pressures it was considered that the core is confined by internal and external stirrups, both supposed to have yielded. Moreover, uniform confinement pressures were reduced by taking into account a reduction in the effectively confined area both in plant and in elevation.

The two parts of the lateral confining pressure were calculated by imposing the equilibrium condition on the stirrup plane; consequently the uniform confining pressure has to equilibrate the tensile force on the lateral legs of the stirrup, as shown in Fig. 3, and the two components of the total confining pressure can be written as follow:

$$f_{l,c} = \frac{2 \cdot f_{ysc} \cdot A_{stc}}{(b - c_c) \cdot s_c} \quad \text{Due to internal stirrups} \tag{5}$$

$$f_{l,j} = \frac{2 \cdot f_{ysj} \cdot A_{stj}}{(B - \delta) \cdot s_j} \quad \text{Due to external stirrups} \tag{6}$$

A_{stc} and A_{stj} being the area of the legs in the core and jacket stirrups.

To take into account the non-uniformity of the confinement pressure both in plant and in elevation because of the discontinuities of the stirrups, an effectiveness coefficient was introduced and calculated as in Mander et al. [16]. Specifically, the effective and ineffective confined concrete cores are calculated as shown in Fig. 4. The effect in plant of the internal stirrups proves to be the following:

$$k_{pc} \cong \left(1 - \frac{4}{6} \cdot \frac{(b - 2 \cdot c_c - 2 \cdot d_{bc})^2}{(b - 2 \cdot c_c)^2}\right) \cong \frac{1}{3} \tag{7}$$

and in elevation:

$$k_{vc} = \left(1 - \frac{s_c}{2 \cdot (b - c_c)}\right)^2 \tag{8}$$

Analogously in plant the effect of the external stirrups (jacket) on the core was calculated as follows:

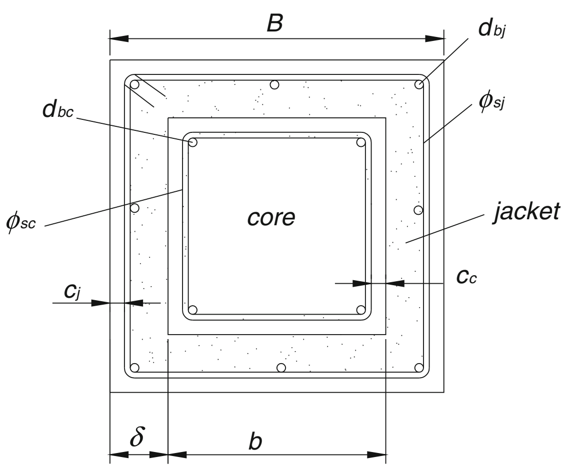


Fig. 2 RC columns externally jacketed

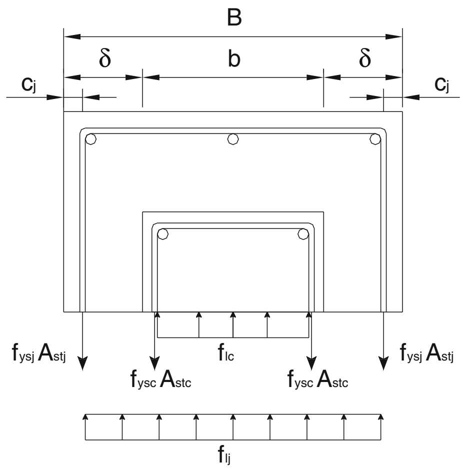


Fig. 3 Equilibrium of the transverse cross-section and calculation of confinement pressure

$$k_{\mathrm{ej}} \cong \frac{b^2 - 4 \cdot \left[\int_{x_1}^{x_2} y(x)\mathrm{d}x - (\delta - c_{\mathrm{j}}) \cdot (x_2 - x_1)\right]}{b^2} = 1 - \frac{2}{3 \cdot b^2} \cdot \sqrt{(b + 2c_{\mathrm{j}} - 2\delta)^3} \cdot \sqrt{b - 2c_{\mathrm{j}} + 2\delta} \leq 1 \quad (9)$$

for $\frac{\delta}{b} \leq \frac{1}{2\left(1 - \frac{c_{\mathrm{j}}}{\delta}\right)}$

and in elevation:

$$k_{\mathrm{vj}} = \left(1 - \frac{s_{\mathrm{j}}}{2 \cdot (b + 2\delta - 2c_{\mathrm{j}})}\right)^2 \quad (10)$$

Figure 5 shows the variation in the k_{ej} coefficient given by Eq. (9) with the variation in δ/b for two different values of the cover-to-jacket thickness ratio (c_{j}/δ). It is interesting to observe that with the δ/b ratio between 0.13 and 0.33, which are the values generally adopted, the effectiveness coefficients are between 0.35 and 0.7.

The equivalent confinement pressure was obtained considering the effects of internal and external stirrups separately, giving the following:

$$f_{\mathrm{l,core}} = \frac{2 \cdot f_{\mathrm{ysc}} \cdot A_{\mathrm{stc}}}{(b - c_{\mathrm{c}}) \cdot s_{\mathrm{c}}} \cdot \left(1 - \frac{4}{6} \cdot \frac{(b - 2 \cdot c_{\mathrm{c}} - 2 \cdot d_{\mathrm{bc}})^2}{(b - 2 \cdot c_{\mathrm{c}})^2}\right) \cdot \left(1 - \frac{s_{\mathrm{c}}}{2 \cdot (b - c_{\mathrm{c}})}\right)^2 + \frac{2 \cdot f_{\mathrm{ysj}} \cdot A_{\mathrm{stj}}}{(B - \delta) \cdot s_{\mathrm{j}}} \cdot \left[1 - \frac{2}{3 \cdot b^2} \cdot \sqrt{(b + 2c_{\mathrm{j}} - 2\delta)^3} \cdot \sqrt{b - 2c_{\mathrm{j}} + 2\delta}\right] \cdot \left(1 - \frac{s_{\mathrm{j}}}{2 \cdot (b + 2\delta - 2c_{\mathrm{j}})}\right)^2 \quad (11)$$

The ultimate strain $\varepsilon_{\mathrm{cu}}$ of the confined concrete was assumed as in Penelis and Kappos [18] considering both the effects of internal and external stirrups in the following form:

$$\varepsilon_{\mathrm{cu}} = \varepsilon_{\mathrm{co}} + \frac{2.8}{f_{\mathrm{cc}}} \cdot \left[\frac{\varepsilon_{\mathrm{suj}} \cdot A_{\mathrm{sj}}}{s_{\mathrm{j}} \cdot (B - \delta)} + \frac{\varepsilon_{\mathrm{suc}} \cdot A_{\mathrm{sc}}}{s_{\mathrm{c}} \cdot (b - c_{\mathrm{c}})}\right] \quad (12)$$

To avoid cracking in tension of the concrete jacket causing premature yielding of stirrups of the jacket (rendering ineffective the confinement induced in the inner core), the following condition should be satisfied:

$$\delta \cdot s_{\mathrm{j}} \cdot f_{\mathrm{ct}} = A_{\mathrm{stj}} \cdot f_{\mathrm{ysj}} \quad (13)$$

If the tensile strength of the concrete jacket is assumed as

$$f_{\mathrm{ct}} = 0.56 \cdot \sqrt{f_{\mathrm{cj}}} \quad (14)$$

With f_{cj} the compressive strength of concrete jacket and introducing the transverse steel ratio of the jacket defined as

$$\omega_{\mathrm{st}} = \frac{2 \cdot A_{\mathrm{stj}}}{b \cdot s_{\mathrm{j}}} \cdot \frac{f_{\mathrm{ysj}}}{f_{\mathrm{c0}}} \quad (15)$$

From Eq. (13) utilising Eqs. (14) and (15) we obtain:

$$\omega_{\mathrm{st}} \geq 1.12 \cdot \frac{\delta}{b} \cdot \frac{f_{\mathrm{cj}}}{f_{\mathrm{c0}}} \quad (16)$$

Figure 6 shows ϖ_{st} as a function of the δ/b ratio for two different designs concrete strengths of the jacket (10 and 40 MPa) and for a fixed compressive strength of the unconfined core (10 MPa). From the graph it emerges that in order to avoid early failure of the external layer; a minimum amount of transverse reinforcement should be provided in the jacket. For the most commonly adopted jacket thicknesses of 40 and 100 mm, the transverse reinforcement ratio should be between 0.05 and 0.12 (valid for a concrete strength of the jacket equal to 40 MPa). This means that for a member with side 300 mm, reinforced with transverse stirrups having 10 mm diameter and 391 MPa in yield stress, the corresponding pitches should be respectively equal to 400 and 170 mm. These values are generally respected if the stirrup spacing adopted in the jacket is chosen as the half-side of the inner column. However, it should be noted that lower values of the pitch can be needed in order to avoid the longitudinal bar buckling after the cover is

Fig. 4 Effectively confined core for jacketed *square* cross-section

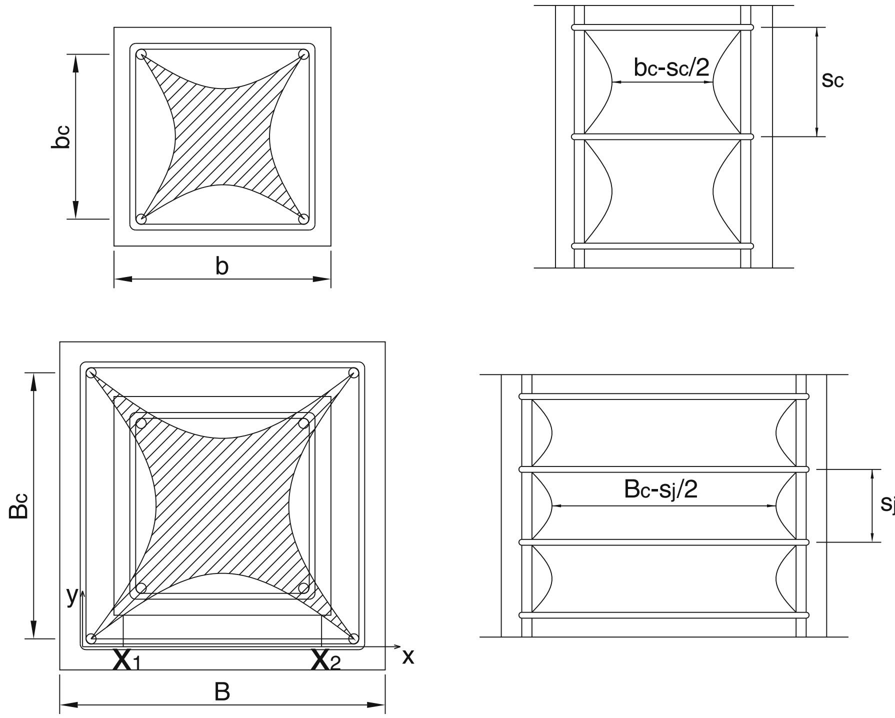
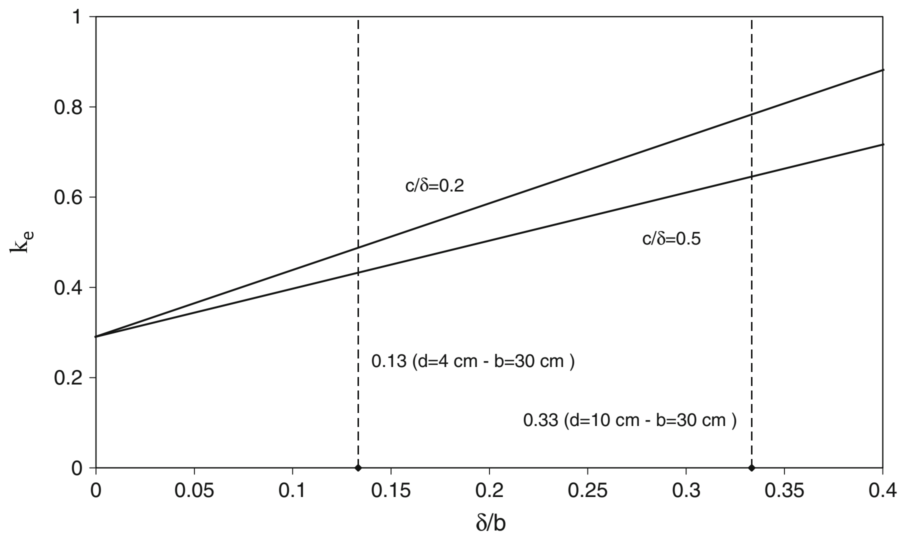


Fig. 5 Variation in k_e with δ/b ratios



spalled off, and consequently the values provided should be checked.

4 Stress–strain curves for steel in tension

For transverse steel with strain hardening behavior a three-linear-strain hardening model is assumed, as suggested in Dhakal and Maekawa [8], in the following form:

$$\sigma_s = \begin{cases} \varepsilon_s \cdot E_s & \text{for } \varepsilon_s \le \varepsilon_y \\ f_y & \text{for } \varepsilon_y \le \varepsilon_s \le 8 \cdot \varepsilon_y \\ f_y + (\varepsilon_s - \varepsilon_y) \cdot E_p & \text{for } 8 \cdot \varepsilon_y \le \varepsilon_s \le 40 \cdot \varepsilon_y \end{cases} \tag{17}$$

E_p being the hardening modulus of the bilinear law and assumed to be equal to $0.03E_s$.

5 Stress–strain curves for steel in compression

It has to be noted that considering the sectional analysis, the impact of compressed steel contribution in a large section could be negligible. However if the bar buckles for a critical length longer than the pitch of the stirrups, confinement effects are loosen and sudden failure occurs. Hence an accurate definition of the stress–strain law of steel in compression is needed.

It has to be stressed that concrete jacketing improves compressed bars behaviour (reduction of critical length) due to the coupled effects of concrete external RC jacket and also to the stirrups (see [6]. Moreover, beneficial effect on critical length are due both to the external layer of steel bars and to the bars of the concrete core.

The constitutive law assumed for a compressed longitudinal bar is given by Eq. (17), neglecting buckling effects. If buckling effects are considered, the average steel bar compressive stress–strain curve as written in Dhakal and Maekawa [8] is assumed here in the form:

$$\frac{\sigma}{\sigma_1} = 1 - \left(1 - \frac{\sigma^*}{\sigma_1^*}\right) \cdot \left(\frac{\varepsilon - \varepsilon_y}{\varepsilon^* - \varepsilon_y}\right) \quad \text{for } \varepsilon_y < \varepsilon \leq \varepsilon^*$$
$$\sigma \geq 0.2 \cdot f_y; \quad \sigma = \sigma^* - 0.02 \cdot E_s \cdot (\varepsilon - \varepsilon^*) \quad \text{for } \varepsilon > \varepsilon* \tag{18}$$

with

$$\frac{\varepsilon^*}{\varepsilon_y} = 55 - 2.3 \cdot \sqrt{\frac{f_y}{100}} \cdot \frac{L}{d_b}; \quad \frac{\varepsilon^*}{\varepsilon_y} \geq 7$$
$$\frac{\sigma^*}{\sigma_1^*} = \alpha \cdot \left(1.2 - 0.016 \cdot \sqrt{\frac{f_y}{100}} \cdot \frac{L}{d_b}\right); \quad \frac{\sigma}{f_y} \geq 0.2 \tag{19}$$

α being 1 for linear hardening bars and 0.75 for perfectly elastic–plastic bars, and L being the buckling length. Figure 7 shows typical stress–strain curves for compressed bars, including the definition of symbols used in Eqs. (18) and (19).

The length L utilized in the Dhakal and Maekawa [8] model is assumed here as the critical length as in Campione [6]. With reference to the bar placed at the corner, the stiffness of the stirrup (E_s is the elastic modulus of steel before yielding and in the plastic range is the reduced modulus assumed here to be 0.03 E_S) proves to take the form:

$$\alpha_{sj} = \frac{2 \cdot E_s \cdot \pi \cdot \varphi_{sj}^2}{4 \cdot (B - \delta)} \quad \text{(external stirrup)}$$
$$\alpha_{sc} = \frac{2 \cdot E_s \cdot \pi \cdot \varphi_{sc}^2}{4 \cdot (b - c)} \quad \text{(internal stirrup)} \tag{20}$$

Analogously, with reference to the bar placed along the side of the transverse cross-section, the stiffness of the system (in this case, the stiffness represents the force necessary to produce a unit displacement along the direction perpendicular to the leg of the stirrups) at

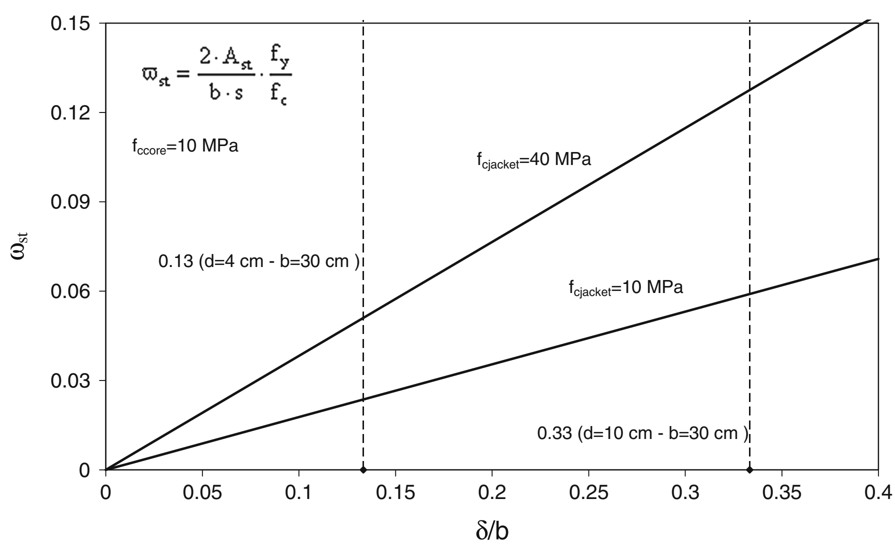


Fig. 6 Required mechanical ratios of external stirrups with δ/b variation

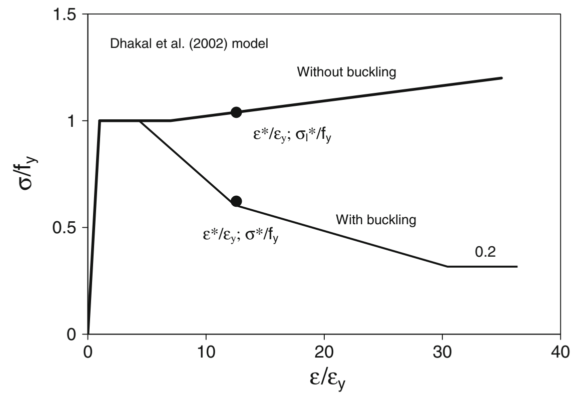


Fig. 7 Stress–strain *curves* adopted for compressed *bars* according to Dhakal et al. (2002)

first yielding of the transverse steel is:

$$\alpha_{sj} = \frac{48 \cdot \pi \cdot \varphi_{sj}^4}{64 \cdot (B - \delta)^3} \cdot E_s \quad \text{(external stirrup)}$$

$$\alpha_{sc} = \frac{48 \cdot \pi \cdot \varphi_{sc}^4}{64 \cdot (b - c)^3} \cdot E_s \quad \text{(internal stirrup)} \tag{21}$$

Using the continuum approach, diffused springs can be assumed by introducing a fictitious parameter $k = \frac{\alpha_{eq}}{s}$ representing the stiffness per unit length.

In this connection, a problem of an elastic beam on elastic springs subjected to an axial compressive load allows one to determine the critical load and the length L involved in the buckling phenomena of longitudinal bars.

In this case, as proposed in Russo and Terenzani [19] the critical length L is expressed by

$$L = 2 \cdot \pi \cdot \left(\frac{E_r \cdot I}{3 \cdot k}\right)^{1/4} = 4.77 \cdot \left(\frac{E_r \cdot I}{k}\right)^{1/4} \quad \text{(in mm)} \tag{22}$$

E_r being the reduced modulus proposed by Papia and Russo [17] in the form:

$$E_r = E_s \cdot \left[2.13 \cdot \left(\frac{E_h}{E_s}\right)^{0.89} - 4.11 \cdot \left(\frac{E_h}{E_s}\right)^2\right] \tag{23}$$

and I the moment of inertia of the longitudinal bars assumed as $I = \frac{\pi \cdot d_b^4}{64}$ for bars in the core and in the jacket (with d_b assumed d_{bc} or d_{bj} for jacket or core bars), while E_h is the hardening modulus, which can be assumed as 0.03 E_s in simplified manner.

Figure 8 shows the variation in the critical length L, dimensionless with respect to the bar diameter (jacket or core), with respect to the s_j/b ratios. The case of a corner bar of the core at constant pitch $s = 14\ d_b$ was considered. In addition, the case of corner bars of jackets with stirrups of diameter 8 and 10 mm were analyzed with the variation in the s_j/b ratios. The results obtained show that by using very stiff stirrups of close pitch reduced slenderness is achieved. In other cases, softening behavior of compressed bars is expected after the cover is spalled off and transverse stirrups have to have yielded.

It has to be stressed that the critical length L is chosen considering only the stirrups (internal and external) contributions calculated at their yielding and verifying that the pitch of stirrups is enough low to ensure that no buckling of compressed bars occurs. Consequently: the critical length is the maximum; the concrete core can be considered effectively confined; the cover contribution can be included. However, if these conditions are not verified, as in the case of HSC buckling of compressed bars occurs and the contribution of confinement on the concrete core and that of the concrete cover should be not included.

6 Comparison between analytical and experimental response for strengthened members

In this section, on the basis of the model adopted for the constituent materials given in the previous section, the complete load-axial shortening curves of compressed columns, the moment–curvature and resistance domains are derived and compared with the available experimental data. To plot the complete stress–strain curves in compression the procedure adopted in Badalamenti et al. [2] was utilised.

The moment capacity of strengthened columns was computed analytically using the well-known fibre method adopted for discretization of the cross-section implemented in SAP 2000 [7]. Sections analyzed in the following sections 8 strips for the jackets (higher than the number of strips generally adopted to model the concrete cover of R.C. cross-section) and 40 for the confined region.

In the hypothesis of a plane section and considering the cross-section subjected to the coupled effects of axial force and bending moment, the moment-axial

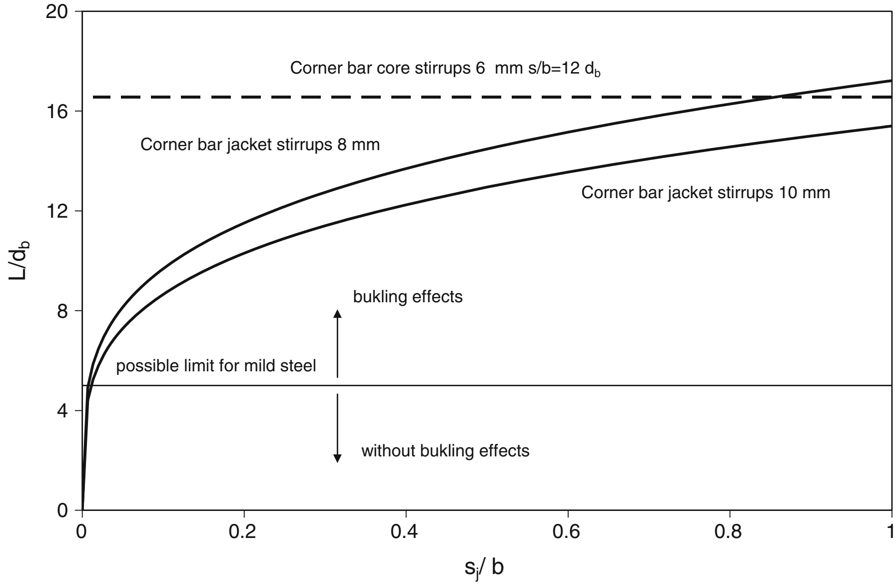


Fig. 8 Variation in critical length with *s*/*b* ratios

force domain and the moment–curvature diagrams for fixed values of axial force were here derived. The cross-section was discretized considering the concrete core and concrete jacketing as confined and unconfined concrete. The tensile strength of the concrete is not considered. For simplicity's sake, the second-order effects for the whole column are neglected.

6.1 Members subjected to compression

The data used for comparison for members in compression are those of Ersoy et al. [9] and Takeuti et al. [21]. The two researches refer to the use of external cages constituted by concrete jacketing of normal and high strength concrete, respectively.

The data given in Ersoy et al. [9] refer to specimens with square cross-section of dimensions 130 × 130 × 650 mm with four longitudinal bars of 10 mm in diameter and stirrups 4 mm in diameter and spaced at 40 mm. The columns were strengthened by concrete jacket of thickness 30 mm reinforced with four longitudinal bars of 10 mm in diameter and stirrups 4 mm in diameter and spaced at 40 mm.

The steel type for RC columns and concrete jacket had minimum yield stress of 280 MPa and the concrete used in the columns had compressive strength of 28 MPa for the core and 23 MPa for the jacket. Maximum aggregate size was 10 mm. Columns here utilised for comparison were type M and LS that according to Ersoy et al. [9] were monolithic and strengthened during the loading process. Specimens LS reached 95 % of the load carrying capacity of monolithic member.

The data given in Takeuti et al. [21] refer to specimens with square cross-section of dimensions 120 × 120 × 900 mm with four longitudinal bars of 8 mm in diameter and stirrups 6.3 mm in diameter and spaced at 90 mm. The columns were strengthened by concrete jacket of thickness 40 mm reinforced with four longitudinal bars of 8 mm in diameter and stirrups 5 mm in diameter and spaced at 70 and 50 mm (specimens S2N and S3N as in [21].

The steel type for S2N specimens (nomenclature of [21] had minimum yield stress of 652 MPa and the concrete used in the columns had compressive strength of 32.7 MPa for the core and 80 MPa for the jacket. Stirrups of jacket had tensile strength of 724 MPa.

The steel type for S3N specimens (nomenclature of [21] had minimum yield stress of 652 MPa and the concrete used in the columns had compressive strength of 24.8 MPa for the core and 81.9 MPa for the jacket. Stirrups of jacket had tensile strength of 724 MPa.

Columns here utilised for comparison were type M and LS that according to Ersoy et al. [9] were monolithic and strengthened during the loading process. Specimens were preloaded and therefore tested in compression.

Figure 9 shows the comparison between the experimental and the analytical response of compressed columns for both cases considered in Ersoy et al. [9] and Takeuti et al. [21]. The comparison shows good agreement in both cases examined and also stresses that the use of high strength jackets allows to obtain very high increases in load-carrying capacity, but more brittleness is also observed.

This fact can be observed with particular reference to specimens tested by Takeuti et al. [21] (Fig. 9b); the latter as discussed above were characterized by a jacket with δ/b ratio equal to 0.33 and concrete compressive strength of about 80 MPa. It has to be noted that High Strength concretes, such that used for the jacket, are characterized by a quasi-linear behaviour until the peak and a marked slope in the softening phase. Consequently the behaviour of columns is characterized by a linear trend and by a drop in capacity after that the peak was reached. Afterwards the effect of the jacket could be negligible and the axial capacity of the member is mainly due to the core. In this example the loss of confinement pressure after the concrete cover spalling was not considered, because it is negligible. The comparison with experimental results proves to be acceptable and that confirms that this assumption is adequate.

6.2 Cross-section analyses

For cross-section analyses a classic approach based on the use of discretization into strips at constant strain of the cross-section is made. The constitutive laws adopted are those described in the previous section. Of course the impact of each constituent in a global analysis is an important aspect to be considered. But at this stage of research is it out of the scope of the paper and only comparison with existing experimental data is made.

The validation of the proposed model was carried out with the data of Ersoy et al. [9] and Sfakianakis [20]. The loading conditions considered were fixed level of axial force and/or fixed value of eccentricity.

The data of Ersoy et al. [9] relative to specimens MM (monolithic) and SR (repaired) refer to a column with the following: $b = 160$ mm, $\delta = 35$ mm, $f_c = 27$ MPa for MM and 40.3 MPa for SR; $f_{cj} = 33$ MPa, n.4 12 mm longitudinal bars for core and jacket, stirrups having 4 mm diameter for core at pitch 100 and 8 mm at pitch 100 mm for jacket. The longitudinal bars had yielding stress 300 MPa and the stirrups had 260 MPa yielding stress.

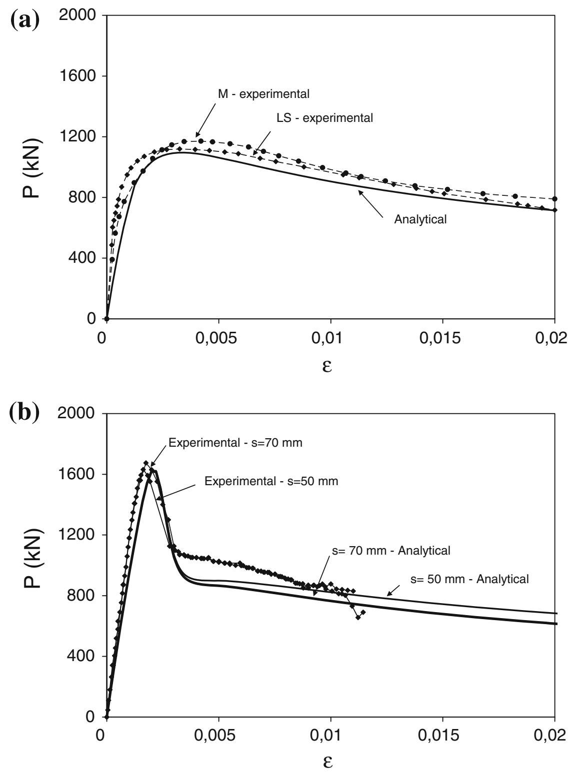


Fig. 9 Comparison of experimental results (load shortening *curve*) and analytical prediction obtained with the proposed model: **a** data of Ersoy et al. [9]; **b** data of Takeuti et al. [21]

Sfakianakis [20] proposed a numerical procedure based on computer graphics able to determine the strength domains. Among the different examples used to validate the numerical procedure, one refers to a rectangular RC columns of sides 300 × 350 mm having a RC jacket of 50 mm. The strength of the core was 16 MPa, and that of the jacket 25 MPa. For the core four 16 mm longitudinal bars with 220 MPa of yield stress were adopted, while for the jacket 12 bars having 14 mm diameter and yield stress 400 MPa were adopted. The stirrups of the core were 4 mm at pitch for MM and 40.3 for SR; $f_{cj} = 33$ MPa, 4 12 mm longitudinal bars for core and jacket, stirrups having 8 mm at pitch 100 mm and for the jacket 10 mm at pitch 100 mm. The stirrups had 260 MPa yielding stress.

Figure 10 shows the experimental and theoretical moment–curvature diagram obtained for a fixed level of axial force of 500 kN in the case of Ersoy et al. [9].

Figure 11 shows the moment-axial force domain obtained with the current model and with the

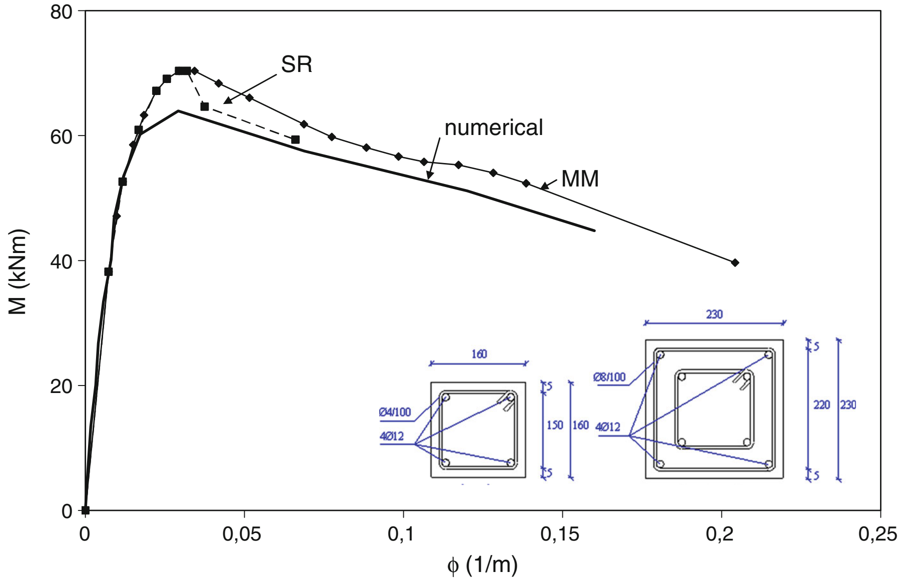


Fig. 10 Comparison of experimental results with moment–curvature diagrams obtained with the current model (Data of Ersoy et al. [9])

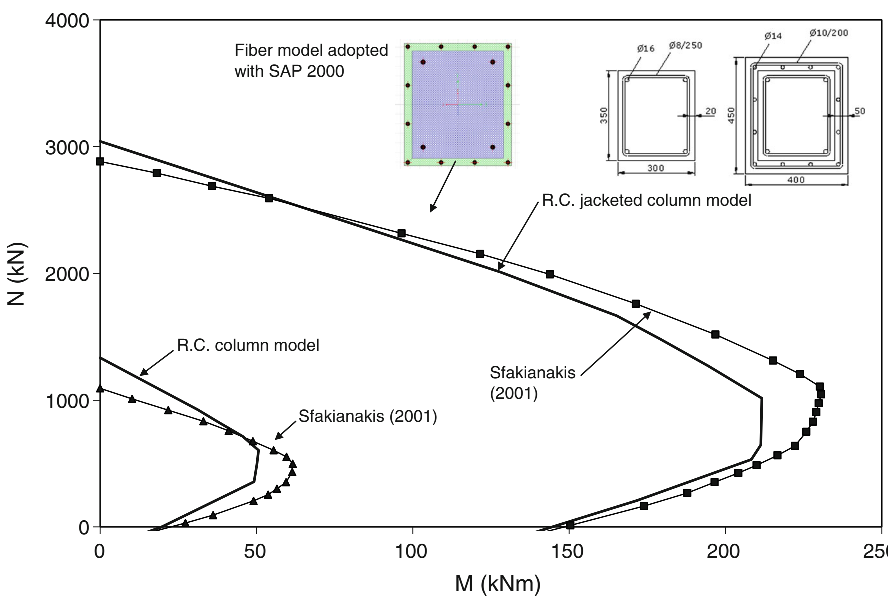


Fig. 11 Moment axial force domain (data of [20]

theoretical procedure derived by Sfakianakis [20]. In both cases the comparison is satisfactory in terms of both peak load and maximum curvature and moment axial force domain.

6.3 Numerical examples

In this section two numerical examples are shown. The examples refer to an externally jacketed RC column, compressed member and to a cross-section analysis of strengthened members under axial load and bending moment.

The data utilised refer to a square cross-section of side 300 mm and a jacket of 50 mm. The core strength was 18 MPa, and the jacket strength 25 MPa for type 1 and 60 MPa for type 2. For the core four 12 mm longitudinal bars with 280 MPa yield stress were adopted, while for the jacket 4 bars having 12 mm

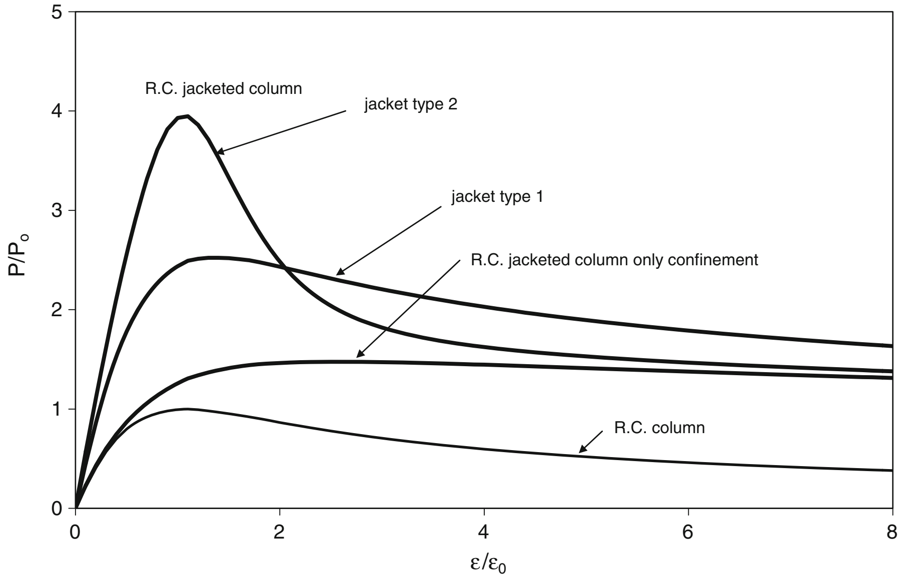


Fig. 12 Load axial–strain *curves* for normal and high strength jacket

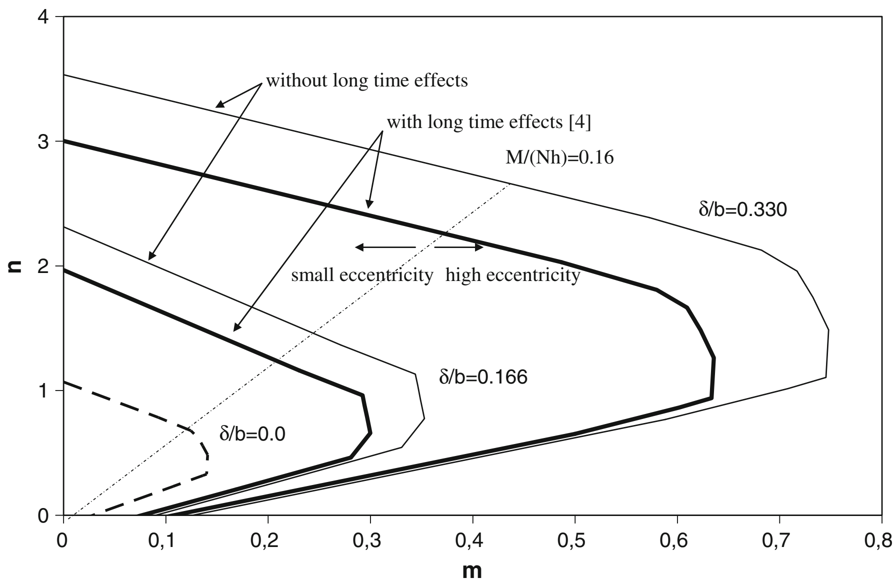


Fig. 13 Effect of thickness of concrete jacket on moment axial force domain

diameter and yield stress 450 MPa were adopted. The stirrups of the core were 8 mm at pitch 200 mm and had yielding stress 280 MPa, while the stirrups of the jacket had 8 mm in diameter at pitch 100 mm and yielding stress 450 MPa. The design strength was assumed using the safety factors given by Eurocode 2 [10]. For members under axial load and bending moment the concrete jacket had 25 MPa compressive strength and different thicknesses (50 and 100 mm, respectively). The choice of these parameters was related to a possible situation of existing columns designed only for gravity load, to be strengthened using the concrete jacket technique.

Figure 12 gives the compressive response of the columns and of the single constituents (steel bars, core, and jacket). The comparison highlights the fact that by using a concrete jacket very high strength increases in the load-carrying capacity are achieved. If high strength jacket is utilised the highest strength is obtained but overall brittle behaviour is expected. This highlights the importance of the accurate choice of the grade of concrete jackets.

Figure 13 shows the moment axial load diagram for the cases examined. Also in this case curves of monolithic members were penalized by the 0.85 factor assumed in the Greek Code (GRECO) and mentioned in Vandoros and Dritos [24]. The comparison shows the advantages of using a concrete jacket for the increases in the flexural strength of the member and the effects of absence of connection between old and new concrete and shrinkage.

Adopting a δ/b ratio of 0.16 the axial capacity increases by about 100 %. By contrast, the ultimate bending moment increases about three times with respect to that of an unreinforced member. For a δ/b ratio 0.33 higher increases were observed, but in the opinion of the author this is only a theoretical result because in many cases the load-carrying capacity of strengthened columns has to be limited according to the shear strength of the connected beams.

7 Conclusions

In the present paper an analytical model is derived for the determination of the load axial strain and the moment–curvature response of RC concrete columns externally strengthened with the concrete jacketing technique. The analytical expressions adopted here for the stress–strain response of confined concrete and steel reinforcement (steel bars) are able to include confinement effects induced by internal and external stirrups. The analytical results generated here in terms of load-shortening curves, moment-axial force domain and moment–curvature diagrams were compared with experimental data available in the literature and with those obtained using the existing analytical models. The comparison shows good prediction of the experimental results. Lastly, a parametric analysis was carried out to highlight the influence of the geometrical and mechanical properties of the strengthening devices (strength and thickness of jacket) on the flexural response and ductility resources of strengthened columns.

Acknowledgments This work has benefited from material deriving from the 2010–2013 Research Project ReLUIS (Rete dei Laboratori di Ingegneria Sismica), AT 1, Task 1.1.2: Strutture in Cemento Armato ordinarie e prefabbricate.

References

1. Altun F (2004) An experimental study of the jacketed reinforced-concrete beams under bending. Constr Build Mater 18:611–618
2. Badalamenti V, Campione G, Mangiavillano ML (2010) Simplified model for compressive behavior of concrete columns strengthened by steel angles and strips. ASCE J Eng Mech 136(2):230–238
3. Branco F, Julio ES, Silva VD (2003) Structural rehabilitation of columns with reinforced concrete jacketing. Prog Struct Eng Mater 5:29–37
4. Branco F, Julio ES, Silva VD (2004) Concrete-to-concrete bond strength. Influence of the roughness of the substrate surface. Constr Build Mater 18:675–681
5. Branco F, Julio ES, Silva VD (2008) Reinforced concrete jacketing—interface influence on cyclic loading response. ACI Struct J 105(4):471–477
6. Campione G (2011) Compressive behaviour of short fibrous reinforced concrete members with square cross-section. Struct Eng Mech 37(6):649–669
7. Computers and Structures (2010) SAP2000: integrated software for structural analysis and design CSI, version 14, Berkeley
8. Dhakal RP, Maekawa K (2002) Modelling of post-yield buckling of reinforcement. ASCE J Struct Eng 128(9): 1139–1147
9. Ersoy R, Tugrul TA, Suleiman U (1993) Behavior of jacketed columns. ACI Struct J 90:3
10. Eurocode 2 (2004) Design of concrete structures: part 1-1: general rules and rules for Buildings, EN 1992-1-1
11. Julio ES, Branco E, De Silva V (2001) Structural rehabilitation of columns with reinforced concrete jacketing. Repair Rehabil 5:29–37
12. Julio ENBS, Branco FAB, Silva VD (2004) Influence of the roughness of the substrate surface. Constr Build Mater 18:675–681
13. Julio ENBS, Branco FAB, Silva VD (2005) Reinforcing concrete jacketing-interface influence on monotonic loading response. ACI Struct J 102(2):252–257
14. Lampropoulos A, Dritos S (2010) Concrete shrinkage effect on columns strengthened with concrete jackets. Struct Eng Int 3:234–239
15. Lampropoulos A, Dritos S (2011) Modelling of RC columns strengthened with RC jackets. Earthq Eng Struct Dyn 40: 1689–1705
16. Mander JB, Priestley MJN, Park R (1988) Theoretical stress-strain model for confined concrete. ASCE J Struct Eng 114(8):1804–1826
17. Papia M, Russo G (1989) Compressive strain at buckling of longitudinal reinforcement. ASCE J Struct Eng 115(2): 382–397
18. Penelis GG, Kappos AJ (1997) Earthquake resistant concrete structures. E&FN Spon, London, p 572
19. Russo G, Terenzani L (2006) Non linear buckling model for the longitudinal reinforcement in RC columns. In: studies and research, graduate school in concrete structures, vol. 22. Fratelli Pesenti, Politecnico di Milano, Milan, pp 227–303
20. Sfakianakis MG (2002) Biaxial bending with axial force of reinforced composites, repaired concrete sections of

arbitrary shape by fiber model and computer graphics. Adv Eng Softw 33:227–242

21. Takeuti AR, Bento de Hanai JB, Mirmiran A (2008) Preloaded RC columns strengthened with high-strength concrete jackets under uniaxial compression. Mater Struct 41:1251–1262
22. Thermon GE, Pantazopoulon SJ, Elnashai AS (2007) Flexural behavior of brittle RC members rehabilitated with concrete jacketing. ASCE Struct J 133(1):1373–1384
23. Vandoros KG, Dritos SE (2006) Axial preloading effects when reinforced concrete columns are strengthened by concrete jackets. Repair Rehabil 8:79–92
24. Vandoros KG, Dritos SE (2008) Concrete jackets construction details effectiveness when strengthening RC columns. Constr Build Mater 22:264–276