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Optimal seismic retrofitting of reinforced concrete columns with steel jacketing technique: a pushover-based genetic algorithm approach

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ABSTRACT
Steel jacketing (SJ) of columns is widely employed as seismic retrofitting technique to provide additional deformation and strength capacity to existing reinforced concrete (RC) frame structures. The use of SJ is associated with non-negligible costs depending on the amount of structural and non-structural manufacturing and materials. The paper presents an optimization framework aimed at obtaining the optimal increase of seismic capacity/demand performance with minimization of retrofitting costs. It is shown the case study of a 3D RC frame implemented in OpenSEESand handled within the framework of a genetic algorithm. The algorithm iterates geometric and mechanical parameters configuration in order to match the optimal retrofitting solution based on static pushover analysis outcomes. Results of the proposed framework will provide optimized location and amount of steel-jacketing reinforcement, showing that the use of engineering optimization techniques can be effectively used to reduce retrofitting costs substantially unaltering structural safety.

1 INTRODUCTION

Retrofitting of reinforced concrete columns by cages arranged by steel angles and battens (steel jacketing) is a widely employed technique to improve strength and deformation capacity of beams and columns of existing buildings presenting critical conditions with respect to seismic and gravity loads. Steel jacketing of columns can be generally arranged in two ways. The first provides a moment resisting connection between the steel cages and the slabs. In this case, besides the confinement action exerted by the cage, additional flexural strength is provided. Since moment resisting connection are not always easy to relize, steel jacketing is often arranged by simply providing the cages. Even in this case a certain additional flexural resistance is observed because of friction forces transfer between the steel angles and the concrete column (Campione et al. 2017), but the most significant contribution is related to the increase of deformation capacity as consequence of the strong confinement action. Experimental and numerical investigation have been carried out both for the first typology of arrangement (Braga and Gigliotti 2006, Montuori and Piluso 2009, Nagaprasad et al. 2009, Tarabia 2014) and for the second one (Adam et al. 2007, 2009, Calderon et al. 2009, Badalamenti et al. 2010, Campione et al. 2017).

Despite its effectiveness in providing additional strength and deformation capacity to RC members, it should be said that steel jacketing is an invasive strengthening technique. In fact the reinforcement of column provides also the demolition and reconstitution of eventual portions of masonry infills and plaster. This is associated with significant direct costs and noticeable downtime for the building. A second issue regards the design of the intervention in terms of individuation of the columns to retrofit and the choice of the battens area and spacing. In fact, when approaching by non-linear static analysis (pushover), as a method to assess the performance before and after the intervention, a significant number of attempt iterations are needed to individuate the most suitable retrofitting configuration, especially when the number of columns is large. In absence of a specific optimization process this generally brings the designer to adopt overall compromise solutions which allow obtaining effective seismic performance without an optimization of the costs.

Structural optimization is widely recognized as a valuable computational tool allowing engineers identifying cost-effective designs. A number seismic design optimization applications for steel and reinforced concrete structures (e.g. Liu et al. 2003, Zou et al. 2007, Greco and Marano 2011, Mitropoulou et al. 2011, Akin and Saka 2015, Papavasileiou and Charmpis 2016) are presented in the literature. On the other hand, the issue of the optimization of strengthening and retrofitting interventions for RC structures has not been investigated many times in the past and available studies are limited to the optimization of carbon fiber reinforcement of concrete slabs (Chaves and Cunha 2014) or FRP jackets (Chisari and Bedon 2016), while of steel jacketing reinforcement has never been faced within an optimization framework. Based on the aforementioned premises this paper presents an optimization framework aimed at the determination of the optimal configuration of the steel jacketing reinforcement of columns of reinforced concrete buildings in terms of reinforcement location and spacing between steel battens. The optimization framework makes use of the Matlab ® genetic algorithm (GA) as optimization tool, automatically interfaced with a 3D model realized in OpenSEES (McKenna et al. 2000). The optimization process will show the minimization of the retrofitting costs, driven by the results of the pushover verification (N2 method, Fajfar 2000) in terms of feasibility of the solution obtained for the single generated individuals.

2 THE CASE STUDY STRUCTURES

2.1 *Geometric and material properties*

The case study building consist of a five-storey two-bays reinforced concrete structure designed to resist only to gravity loads.

The structure (Fig. 1a) has polar symmetry in plan and infills contribution is not considered Dimensions in plan are represented in Fig. 1b as well as dimensions of beams and columns. The structure is supposed to be arranged with poor resistance concrete having average unconfined strength f_{c0m}=20 MPa. Steel rebars have average yielding strength f_y=455 MPa. Reinforcement details of beams and columns are listed in Table 1.

The building is supposed being located in Cosenza, soil type C. The reference nominal life V_N is of 100 years. The resulting return period is T_R=975 years.

Table 1. Reinforcement details of beams and columns.

RC members	**b** x **h** (mm)	Longitudinal reinforcement	Transverse reinforcement
Beams	400 x 500	4+4 ϕ18	ϕ6 / 200 mm
Columns	500 x 500	10 ϕ18	ϕ6 / 200 mm

2.2 *OpenSEES model of the RC frame structure*

The 3D RC frame is modelled adopting distributed plasticity force-based elements for beams and columns. A rigid diaphragm constrain is assigned at the nodes of each floor. Vertical loads are assigned by point loads at the top of each column as function of the respective tributary areas in plan. The total weight of each floor is 1440 kN. A “Concrete02” uniaxial material model is attributed to the cross-section fibers. For sake of simplicity it is assumed that the effect of stirrups on concrete confinement is extended to the whole cross-section (Fig. 2). This simplified assumption is used to obtain a formal consistency with the confinement model in the case of concrete confined by stirrups and steel jacketing (Montuori and Piluso 2009) which provides uniform confinement over the cross-section.

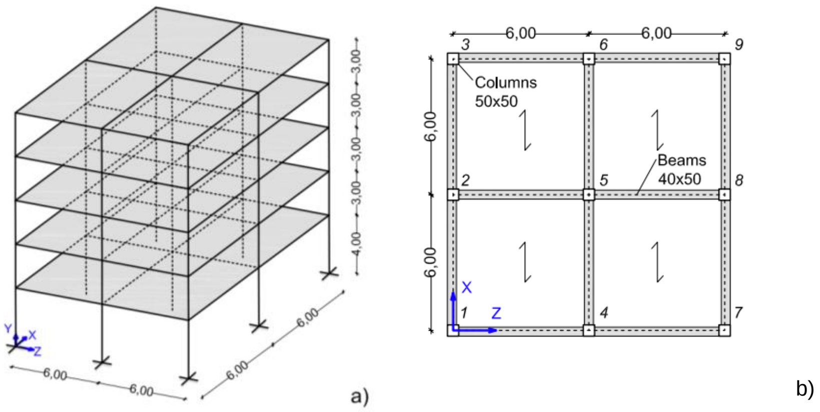

Figure 1. Geometrical dimensions of the case study structures: a) 3D frame view; b) dimensions in plan.

The Concrete02 material is combined with the "MinMax" material in order to observe the crushing of the fiber when the ultimate compressive strain is achieved. Parameters of concrete confined only by stirrups (f_{c0}, f_{cu}, ε_{c0}, ε_{cu}) are evaluated by the Razvi and Saatcioglu (1992) stress-strain model. Steel rebars are modelled using the "Steel02" material model (Giuffrè-Menegotto-Pinto) having yielding stress f_y=455 MPa and hardening ratio b=0.01.

2.3 Modelling of the steel jacketing effect

It is supposed that steel jacketing of columns is arranged without realizing moment resisting connection at the top and the bottom of the columns, while frictional effects (Campione et al. 2017) are neglected. Therefore, the effect of steel jacketing is introduced in the retrofitted columns only as confinement action by simply modifying the stress-strain curve of concrete fibers (Fig. 2). The uniaxial Concrete02 material model is still chosen as stress-strain model for the concrete fibers. The approach by Montuori and Piluso (2009) combined with the with the expressions provided by Razvi and Saatcioglu (1992) are used to evaluate peak (f_{cc},ε_{cc}) and ultimate (f_{ccu},ε_{ccu}) stress and strain values. The model provides the use of a single concrete stress-strain law for the entire section. The lateral confinement pressures $f_{le,x}$ and $f_{le,y}$ along the two direction of the cross-section (Fig. 3) are calculated as:

$$f_{le,x} = k_e \cdot \rho_{st,x} \cdot f_y ; \qquad f_{le,y} = k_e \cdot \rho_{st,y} \cdot f_y \tag{1}$$

in which the evaluation of the transverse reinforcement volumetric ratios $\rho_{st,x}$ and $\rho_{st,y}$ consider both the contribution of internal and external transverse reinforcement as:

$$\rho_{st,x} = \frac{n_{bx} A_{st,x} b_0}{s b_0 h_0} + \frac{2 A_{sb,e} b}{s_b b h} ; \qquad \rho_{st,y} = \frac{n_{by} A_{st,y} h_0}{s b_0 h_0} + \frac{2 A_{sb,e} h}{s_b b h} \tag{2}$$

The coefficient k_e expresses the effectively confined area through the expression:

$$k_e = \left(1 - \frac{s_b - \phi_{st}}{2b_0}\right)\left(1 - \frac{s_b - \phi_{st}}{2h_0}\right) \tag{3}$$

In Eqs. (2) and (3) b is the cross-section base and h its height, $b_0 = b\text{-}2c$ and h_0=$h\text{-}2c$, being c the width of the concrete cover, n_{bx} and n_{by} are the number of stirrups arms along x and y and $A_{st,x}$ and $A_{st,x}$ the respective total areas, ϕ_{st} is the diameter of stirrups, s and s_b are the spacing of the internal hoops and external battens respectively. The term $A_{sb,e}$ represents the mechanically equivalent transverse area of battens and is calculated as:

$$A_{sb,e} = A_{sb} \frac{f_{yb}}{f_y} \tag{4}$$

where A_{sb} is the actual transverse area of battens.

Confined peak stress (f_{cc}) and strain (ε_{cc}) and the ultimate stress (f_{ccu}) and strain (ε_{ccu}) are finally calculated using the expressions provided by Razvi and Saatcioglu (1992). In detail:

$$f_{cc} = f_c + k_1 f_{le} \tag{5}$$

where, with reference to Fig. 3, f_{le} is obtained as:

$$f_{le} = \frac{f_{le,x} b_0 + f_{le,y} h_0}{b_0 + h_0} \tag{6}$$

and is obtained from the Richart's equation as:

$$k_1 = 6.7 f_{le}^{-0.17} \tag{7}$$

The confined peak strain is evaluated as:

$$\varepsilon_{cc} = \varepsilon_c (1 + 5K) \tag{8}$$

where K is:

$$K = k_1 \frac{f_{le}}{f_c} \tag{9}$$

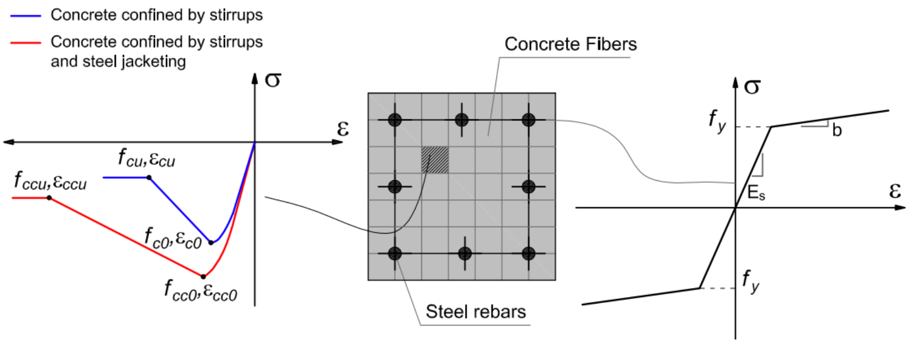

Figure 2. Definition of the fiber cross-section in with and without steel jacketing reinforcement

The determination of ε_{ccu} passes through the evaluation of ε_{cc85}, which is the confined strain associated with the 15% reduction of the peak strength (f_{cc85}). ε_{cc85} is obtained as:

$$\varepsilon_{cc85} = 0.0036 + \rho_s \varepsilon_{cc} \quad (10)$$

in which the original expression of ρ_s is modified as follows to take into account the steel jacketing:

$$\rho_s = \frac{A_{st,x} + A_{st,x} + 4A_{sb,e}}{\tilde{s}(b_0 + h_0)} \quad (11)$$

and $\tilde{s}$ is the ideal average transverse reinforcement spacing defined as:

$$\tilde{s} = \frac{s + s_b}{2} \quad (12)$$

The softening branch is obtained by joining the peak stress-strain point (f_{cc}, ε_{cc}) with the point (f_{cc85}, ε_{cc85}). In the original formulation the stress-strain curve becomes constant after the achievement of a 80% reduction of f_{cc}. For the current case we assume that the crushing of a concrete fiber is attained when f_{cc} is reduced by 30%. Therefore one obtains:

$$f_{ccu} = \alpha f_{cc} \quad (13)$$

with α=0.7 and

$$\varepsilon_{ccu} = \varepsilon_{cc} + \frac{(1-\alpha)(\varepsilon_{cc85} - \varepsilon_{cc})}{0.15} \quad (14)$$

Eqs. (1-14) have been implemented in the TCL OpenSEES script, therefore, any modification of the reinforcement configuration automatically modifies f_{cc}, ε_{cc}, f_{ccu} and ε_{ccu} and is accounted by the Concrete02 uniaxial material during the optimization process.

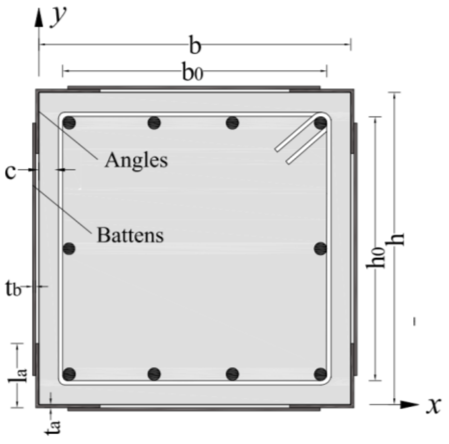

Figure 3. Typical configuration of the cross-section of a column reinforced by steel jacketing.

3 DESIGN OPTIMIZATON FRAMEWORK

3.1 Operating principles and purposes of the optimization framework

Optimization methods based on genetic algorithms, evolution strategies, differential Evolution, etc., have been found to be very effective for structural optimization problems. In the current study, the Matlab ® genetic algorithm (GA) is used in combination with OpeeSEES to minimize an objective function built to compute the retrofitting costs as a function of the defined design variables associated with the steel jacketing reinforcement. The GA works on a population of individuals using a set of operators that are applied to the population. A population is a set of points in the design space representing different possible combinations of the design variables. The initial population is generated randomly by default. The next generation of the population is computed using the fitness of the individuals in the current generation. Each individual represents a combinations of the design variables (for the current case these are the position of the retrofitted columns and the spacing between the battens). The performance of each individual is evaluated by carrying out a pushover analysis and computing the ratio between ductility capacity and demand according to the N2 method (μ_c/μ_d) and the associated retrofitting cost. The retrofitting cost is incremented by a penalty function if the solution is unfeasible (μ_c/μ_d <1). The GA will combine the individuals presenting the better fitness values for each generation. By iterating the computations the GA imitates the evolution from generation to generation of a population (i.e. a group of structural designs) under the imposed constraints.

3.2 Design variables

For the current optimization problem the following simplified assumptions are assumed:

a) The retrofitted columns can be only located within the first the second floor.
b) A uniform pushover lateral forces profile is applied only in one direction;

Moreover the following the following design assumption are made:

a) Steel angles are constituted by L-shaped steel profiles having lateral length l_a=100 mm and thickness t_a=5mm.
b) The thickness of the battens t_b is 5 mm, the width w_b is 50 mm.

c) The minimum and maximum spacings between the battens are 150 and 400 mm respectively.
d) The design yielding strength of steel angles and battens is f_{yb}=275 MPa.
e) The central columns (position 5 in Fig. 1c) at the first and second floor are always retrofitted.

Based on the aforementioned assumptions the design variables are then, the position of the retrofitted column (within the first two floors) and the spacing between the stirrups. The design vector (***b***) can be thus formulated as follows:

$$\boldsymbol{b} = \begin{pmatrix} s_b \\ \boldsymbol{p} \end{pmatrix} \tag{15}$$

where s_b is a scalar belonging to the interval S

$$s_b \in S = [150 \quad 400] \tag{16}$$

while ***p*** is a 16x1 vector collecting the positions of the columns at the first two floor excluding the central ones having the following shape:

$$\boldsymbol{p} = [c_{11} \quad c_{21} \quad c_{31} \quad c_{41} \quad c_{61} \quad c_{71} \quad c_{81} \quad c_{91} \ldots \ldots c_{12} \quad c_{22} \quad c_{32} \quad c_{42} \quad c_{62} \quad c_{72} \quad c_{82} \quad c_{92}\,]^T \tag{17}$$

The elements belonging to ***p*** have the generic shape c_{ij} elements, where i represents the position of the columns with reference to the numbering proposed in Fig. 1c, while j represents the floor. The c_{ij} elements are binary elements assuming the value 0 if the column is not retrofitted and 1 if the column is retrofitted. Therefore c_{ij} elements belong to the binary set named C:

$$c_{ij} \in C = (0 \quad 1) \tag{18}$$

During the optimization process the GA generates the population of individual by assigning the elements of the ***b*** vector. This results in a list of parameters which are read by the TCL script file which builds the model. The pushover analysis is then started and results are processed as described in the subsequent section.

3.3 Processing of pushover results

Pushover curves obtained from each individual are processed in the framework of the N2 method (Fajfar 2000) in order to determine the capacity / demand ductility ratios. The ductility demand (μ_d) can be obtained as:

$$\mu_d = (q^* - 1)\frac{T_C}{T^*} + 1 \quad \text{if } T^* \le T_C$$
$$\mu_d = q^* \qquad \text{if } T^* > T_C \tag{19}$$

where T^* is the period of the equivalent SDOF system (having mass m^*) evaluated from its bilinear capacity curve (Fig. 5):

$$T^* = 2\pi\sqrt{\frac{m^*}{k^*}} \tag{20}$$

and in which, with reference to Fig. 5, k^* is:

$$k^* = \frac{F_y^*}{d_y^*} \tag{21}$$

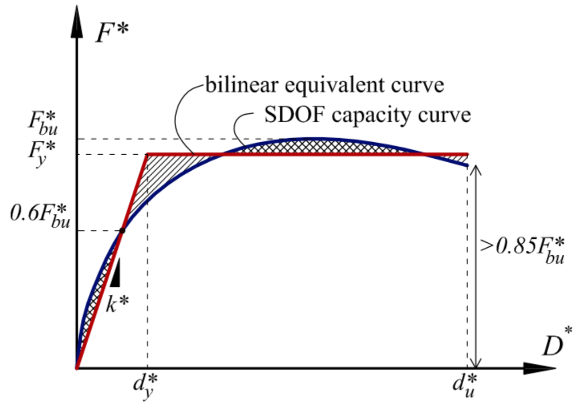

Figure 5. Equivalent SDOF capacity curve and bilinear equivalent curve.

Finally q^* is the reduction factor calculated as the ratio between the force requested to the ideally elastic SDOF and the yielding force:

$$q^* = \frac{S_{ae}(T^*)m^*}{F_y^*} \tag{22}$$

The ductility capacity (μ_c) is instead evaluated as the ratio between the ultimate displacement capacity and the yielding capacity of the bilinear curve:

$$\mu_c = \frac{d_u^*}{d_y^*} \tag{23}$$

The capacity / demand ratio (ξ_μ) is finally defined as:

$$\xi_\mu = \frac{\mu_c}{\mu_d} \tag{24}$$

ξ_μ is the output of the processing of the pushover curves and is used as discriminating factor to make the optimization process understand if the single individual passes the verification check ($\xi_\mu > 1$) or not ($\xi_\mu < 1$).

3.4 The objective function

The objective function monitors the retrofitting costs intended as the material costs and the manpower costs to realize column steel jacketing (C_{SJ}) and necessary works on plasters and masonry (C_M). The general form of the objective is therefore:

$$C = C_M + C_{SJ} \tag{25}$$

In the current study C_M have been estimated considering a fixed cost (c_m) of 2000 € per reinforced column, therefore:

$$C_M = n_c c_m \tag{26}$$

where n_c is the number of retrofitted columns. As regards C_{SJ} , this can be computed as:

$$C_{SJ} = n_c W_{s,T} c_s \tag{27}$$

where W_S is the total weight of steel used to arrange the external cage and c_s is the manpower and material cost per unit weight (estimated in 4.5 €/m^3). The weight of each steel cage can be calculated as:

$$W_{s,T} = (V_{a,T} + V_{b,T})\gamma_s \tag{28}$$

where γ_S is the specific weight of steel, $V_{a,T}$ is the total volume of steel angles applied at the corners of the columns, and $V_{b,T}$ the total volume of battens, which depends of their spacing as follows:

$$V_{b,T} = 4V_b\left(\frac{l_c}{s_b}\right) \tag{29}$$

in which V_b is the volume of a single batten and l_c is the length of the column.

The GA will minimize the retrofitting costs by operating an optimization on the number of retrofitted columns (n_c) and the spacing between the battens (s_b).

3.5 *The penalty function*

The search strategy adopted in GA considers the fitness of a solution and is unaffected by any violation of problem constraints. For the current case the feasibility of a solution is represented by the capacity / demand ratio (ξ_μ), which is determined as illustrated in the previous section. In order to introduce feasibility into fitness of a solution, a penalty function is introduced to take into account the violation of a constraint.

This is simply done by assigning penalization of the fitness values if a solution is not feasible. This can be expressed by changing the objective function (C) into the function F as follows:

$$F = C + \Pi \tag{30}$$

where Π is the penalty function assuming the following values:

$$\Pi = \begin{cases} 0 & \text{if } \xi_\mu \geq 1 \\ C_{max}\left(\dfrac{1}{\xi_\mu}\right)^3 & \text{if } \xi_\mu < 1 \end{cases} \tag{31}$$

where C_{max} is the maximum possible retrofitting cost (reinforcement of all first and second floor columns with s_b=150 mm). Therefore, if a solution if feasible, no penalty is assigned ($F=C$). If a solution is not feasible the current cost is fictitiously increased by C_{max} multiplied by a factor taking into account the distance of the current solution by the feasibility (ξ_μ =1), as also illustrated in Fig. 6.

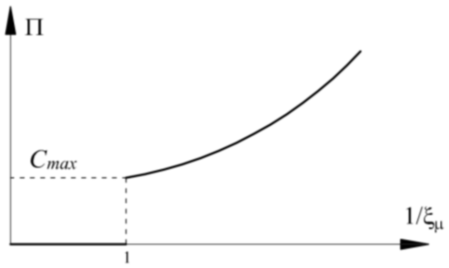

Figure 6. Penalty function.

4 RESULTS OF THE OPTIMIZATION

4.1 *Preliminary tests*

Before starting with the optimization process the structure seismic performance has been tested without any retrofit and under different trial retrofitting configuration. This is done to get some reference point with respect to the final solution obtained at the end of the optimization. The five preliminary test provide the following configurations:

- Test 1: No retrofitting (as built);
- Test 2: Retrofitting of all 1st and 2nd floor column with s_b=150 mm;
- Test 3: Retrofitting of all 1st floor columns and 2nd floor central column with s_b=150 mm;
- Test 4: Retrofitting of all 1st floor and 2nd floor central columns with s_b=250 mm and
- Test 5: Retrofitting of all 1st floor and 2nd floor corner columns and 1st floor and 2nd floor central column with s_b=250 mm.

Results of the tests are illustrated in Figs 6-10 in terms of total base shear and column base shear against top displacement. Results in terms of ductility capacity/demand ratios and retrofitting costs are reported in Table 2.

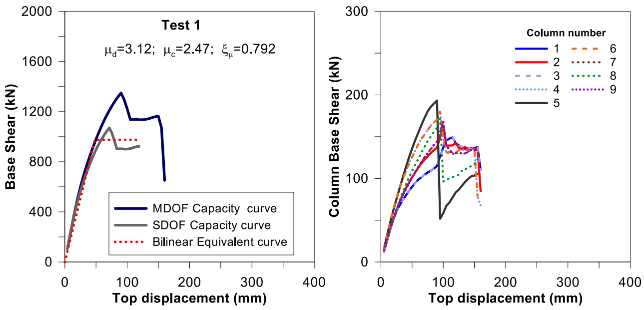

Figure 6. Preliminary Test 1

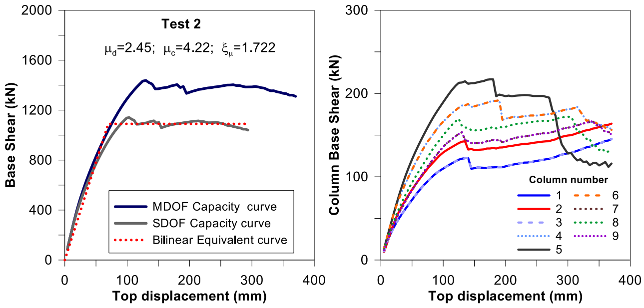

Figure 7. Preliminary Test 2

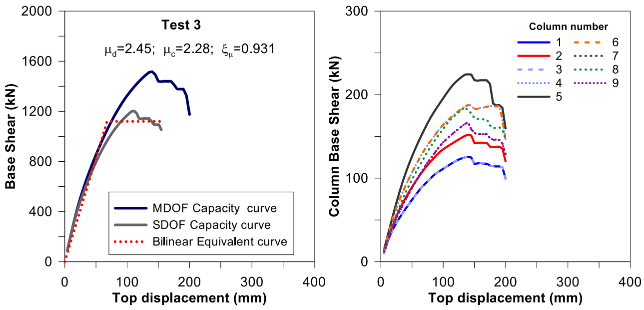

Figure 8. Preliminary Test 3

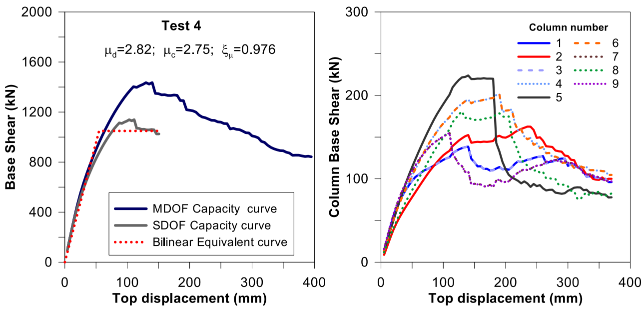

Figure 9. Preliminary Test 4

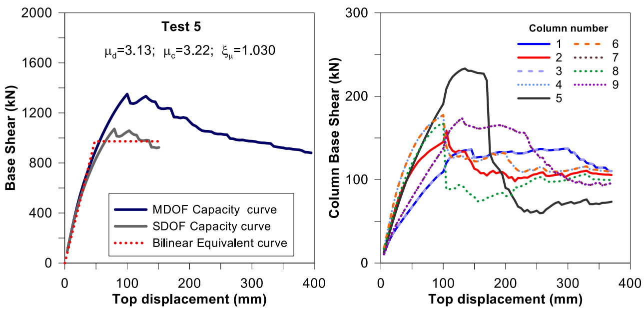

Figure 10. Preliminary Test 5

Table 2. Results of preliminary tests.

Test	μ_d (-)	μ_c (-)	ξ_μ (-)	s_b (mm)	n_c	*C* (€)	**Verif. check**
1	3.120	2.470	0.792	-	0	-	No
2	2.450	4.220	1.722	150	18	51,578.33 €	Yes
3	2.451	2.280	0.931	150	10	28,654.63 €	No
4	2.815	2.748	0.976	250	10	27,170.98 €	No
5	3.125	3.220	1.030	250	10	27,170.98 €	Yes

From the preliminary tests it can be observed that the unreinforced structure (Test 1) has low displacement capacity and suffers a significant load drop (Fig.6) mainly related to the collapse of the central column (5) which carries the largest portion of vertical loads. The overall retrofitting of the first and second floor (Test 2, Fig. 7) significantly improves the response ($\xi_\mu = 1.72$) but is associated with noticeable intervention costs (51,578.33 €). The reinforcement of all the columns of the first floor (Test 3, Fig. 8) is not sufficient to pass the verification check, but it can be observed that more effective results are obtained by retrofitting specific columns at the first and the second floor. In particular, the reinforcement of all the corner columns and the central one has shown to represent a better configuration. In the latter case the structure is retrofitted with an intervention cost of 27,170.98 € obtaining a capacity / demand ratio of $\xi_\mu = 1.03$.

Preliminary results are used to provide a critical overview of the optimization results shown in the following section.

4.2 *Optimization results*

The optimization framework carried out for the reference structure has shown the convergence history illustrated in Fig. 11, where the iterations (generation) are reported against the fitness value obtained from Eq. 30. It can be observed that the algorithm tends to a stable solution after about 900 generations of individuals. Fig. 12 shows the history of the capacity / demand ratio (ξ_μ) values over the generations. It can be noted that the GA only finds feasible solutions after 400 iterations and also that ξ_μ approaches to values close to 1 by going ahead with the iterations. This indicates that optimized solutions are also associated with a major exploitation of the retrofitting intervention.

The optimal solution, found at iteration 1019, provides the retrofitting of a single corner column (column 9) at the first floor besides the central column (column 5) at the first and the second floor with a battens spacing s_b=350 mm.

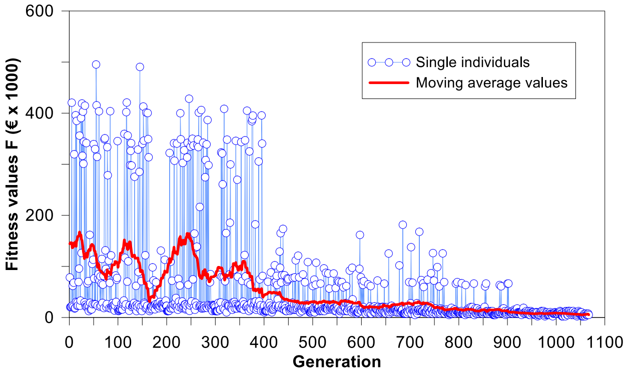

Figure 11. Convergence history of the cost value on the penalized objective function.

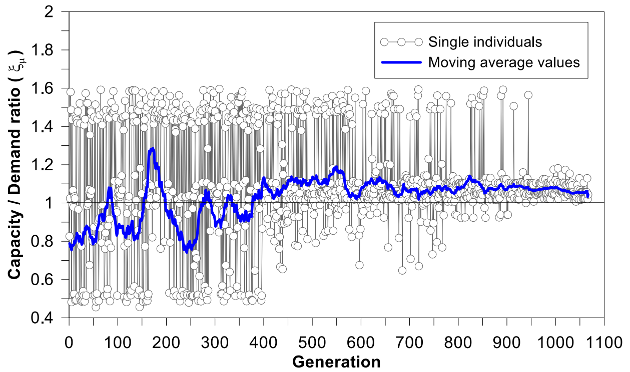

Figure 12. History of the capacity / demand ratio (ξ_μ) values over the generations.

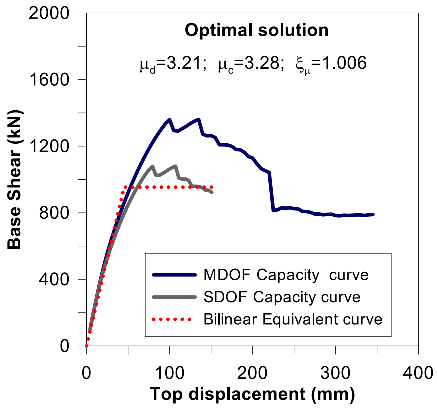

Figure 13. Optimal solution MDOF and SDOF capacity curves.

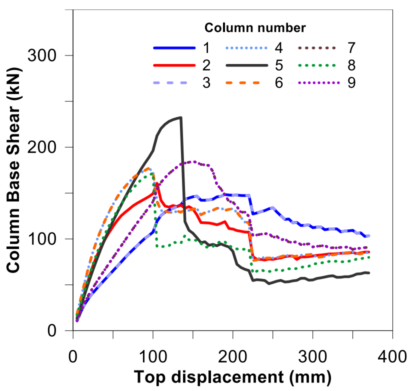

Figure 14. Optimal solution first-storey column capacity curves.

Of course this solution is found considering a pushover force profile acting along *Z* positive direction. Given that the structure has polar symmetry is supposed that all the first storey corner columns will be retrofitted in the same way.

The so defined optimal configuration of reinforcement pushover response is illustrated in Figs. 13-14. The capacity demand ratio finally obtained is ξ_μ =1.006, while the overall cost of the intervention is 15,921.08 €. It is noteworthy observing that the obtained cost is reduced by 40% with respect to the best solution found in the preliminary tests (Test 5). However, in the face of this, obtained ξ_μ factors are almost the same.

5 CONCLUSIONS

The paper presented a framework aimed at the optimization of steel jacketing retrofitting interventions of columns of reinforced concrete frame structures. The method is associated with the adoption of nonlinear static analysis (pushover) as assessment procedure, in the framework of the N2 method. The optimization strategy used a genetic algorithm to minimize the costs of the intervention, operating on the number of reinforced columns and the spacing of battens. The GA is automatically connected with an interactive fiber-section model of the structure realized with OpenSEES. The procedure has been tested in a 5-storey 2-bays reinforced concrete structure. Results have shown that the optimal solution was characterized by a significant reduction of the retrofitting costs, associated with a capacity / demand ratio close to the unit. The current approach has been tested on a very simple frame structure, however, for larger RC structures having a significant number of columns, it expected to get noticeable advantages in terms of economical of and downtime costs.

REFERENCES

Adam, J.M., Ivorra, S., Giménez, E., Moragues J.J., Miguel P., Miragal, C., Calderón, P.A. 2007. Behaviour of axially loaded RC columns strengthened by steel angles and strips, *Steel. Compos. Struct.*, **7**(5), 405–419.

Adam, J.M., Ivorra, S., Pallarès, F.J., Giménez, E., Calderón, P.A. 2009. Axially loaded RC columns strengthened by steel caging: Finite element modelling, *Constr. Build. Mater.*, **23**(6), 2265–2276.

Akin, A., Saka, M.P. 2015. Harmony search algorithm based optimum detailed design of reinforced concrete plane frames subject to ACI 318-05 provisions, *Comput Struct*, **147**, 79–95.

Badalamenti, V., Campione, G., Mangiavillano, M.L. 2010. Simplified model for compressive behaviour of concrete columns strengthened with steel angles and strips, *J. Eng. Mech.*, **136**(2), 230–238.

Braga, F., Gigliotti, M. 2006. Analytical stress-strain relationship for concrete confined by steel stirrups and/or FRP jackets, *J Struct Eng*, **32**(9), 1402–1416.

Calderón, P.A., Adam, J.M., Ivorra, S., Pallarès, F.J., Giménez, E. 2009. Design strength of axially loaded RC columns strengthened by steel caging, *Mater Des*, **30**(10), 4069–4080.

Campione, G., Cavaleri, L., Di Trapani, F., Ferrotto, M.F. 2017. Frictional effects in structural behavior of no end-connected steel-jacketed RC columns: experimental results and new approaches to model numerical and analytical response, *J Struct Eng*, **143**(8), 04017070.

Campione, G., Cavaleri, L., Di Trapani, F., Macaluso, G., Scaduto, G. 2016. Biaxial deformation and ductility doma ins for engineered rectangular RC cross-sections: a parametric study highlighting the positive roles of axial load, geometry and materials, *Eng Struct*, **107**(15), 116–134.

Chaves, L.P., Cunha, J. 2014. Design of carbon fiber reinforcement of concrete slabs using topology optimization, *Con Build Mat*, **73**, 688-698.

Chisari, C., Bedon, C. 2016. Multi-Objective Optimization of FRP Jackets for Improving the Seismic Response of Reinforced Concrete Frames, *American Journal of Engineering and Applied Sciences*, **9**(3), 669-679.

Di Trapani, F., Bertagnoli, G., Ferrotto, M.F., Gino, D., 2018. Empirical equations for the direct definition of stress-strain laws for fiber-section based macro-modeling of infilled frames, *J Eng Mech*, **144**(11): 04018101.

Eurocode 8. 2004. Design of structures for earthquake resistance—Part 1: general rules, seismic actions and rules for buildings, *European Committee for Standardization*, Brussels.

Fajfar, P. 2000. A nonlinear analysis method for performance-based seismic design, Earthq Spectra, **16**, 573–92.

Greco, R., Marano, G.C. 2011. Optimal constrained design of steel structures by differential evolutionary algorithms, *Int J Optim Civil Eng*, **3**, 449–74.

Liu, M., Burns, S.A., Wen, Y.K. 2003. Optimal seismic design of steel frame buildings based on life cycle cost considerations, *Earthquake Eng Struct Dyn*, **32**, 1313–32.

McKenna, F., Fenves, G.L., Scott, M.H., 2000. Open system for earthquake engineering simulation, University of California, Berkeley.

Mitropoulou, C.C., Lagaros, N.D., Papadrakakis, M. 2011. Life-cycle cost assessment of optimally designed reinforced concrete buildings under seismic actions. *Reliab Eng Syst Saf*, **96**, 1311–31.

Montuori, R., and Piluso, V. 2009. Reinforced concrete columns strengthened with angles and battens subjected to eccentric load, *Eng. Struct*, **31**(2), 539–550.

Montuori, R., Piluso, V. 2009. Reinforced concrete columns strengthened with angles and battens subjected to eccentric load, *Eng Struct*, **31**(2), 539–550.

Nagaprasad, P., Sahoo, D.R., Rai, D.C. 2009. Seismic strengthening of R.C. columns using external steel cage, *Earthquake Eng Struct Dyn*, **38**(14), 1563–1586.

Papavasileiou, G.S., Charmpis, D.C. 2016. Seismic design optimization of multi–storey steel–concrete composite buildings, *Comp Struct*, **170**, 49–61.

Razvi, S.R., Saatcioglu, M. 1992. Strength and Ductility of Confined Concrete, J Struct Eng, **125**(3), 281-298.

Tarabia, A.M. 2014. Strengthened of RC columns by steel angles and strips, *Alexandria Eng. J.*, **53**(3), 615–626.

Zou, X.K., Chan, C.M., Li, G., Wang, Q. 2007. Multiobjective optimization for performance ebased design of reinforced concrete frames, *J Struct Eng*, **133**(10), 1462–74.