DESIGN OF SEISMIC UPGRADING OF RC BUILDINGS BY STEEL EXOSKELETON FRAME AND BRBS

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Abstract

During the last decades, seismic upgrading of existing buildings has gained increasing attention by the research community. In fact, in the past many existing buildings were designed for gravity loads only or designed to sustain seismic forces lower than those today expected. Hence, such buildings suffer from serious seismic deficiencies and are not able to sustain the seismic forces prescribed by present seismic codes and zonation. As a consequence, the mitigation of the seismic risk in high seismicity zone, such as the Mediterranean area, and the seismic retrofit of existing buildings has become a pressing need to avoid detrimental human casualties and economic losses.

Out of the retrofit techniques developed by the scientific research, the introduction of Buckling Restrained Braces (BRBs) inside the r.c. framed structure has proved to be effective in upgrading the seismic response of vulnerable existing buildings. Indeed, properly designed BRBs can provide the bare frame with additional stiffness, so that the displacement demand reduces below the displacement capacity of the r.c. frame. However, BRBs inserted within the r.c. frame transmit additional axial forces to the columns, and thus reduce their displacement capacity.

To exploit the dissipative capacity of BRBs and overcome the aforementioned shortcoming, this work proposes the insertion of BRBs inside a steel exoskeleton located outside the r.c. frame. The adoption of the exoskeleton avoids the transmission of the axial forces of BRBs to the r.c. columns. Furthermore, the exoskeleton offers the possibility to integrate the seismic retrofit with energy and technological upgrading of the existing structure and leaves room for innovative architectural solutions as well. This paper develops a design method to size the BRBs and the cross-sections of columns and beams of the external steel exoskeleton for seismic upgrading or r.c. framed buildings. To this end, the proposed design method uses an iterative procedure that reduces the drift demand of the structure below the drift capacity. The design procedure is ruled by three parameters: the design story drift of the r.c. frame, the design ductility of BRBs and the value set for the rate of the drift due to axial elongation of the steel columns.

To investigate the influence of the design parameters on the seismic retrofit design, two r.c. frames with seismic deficiencies are considered as case studies and retrofitted following the proposed design method. The design procedure considered different values of the design parameters. Afterwards, nonlinear dynamic analyses were performed to evaluate the seismic response of the upgraded frames and compare it with that of the bare frames.

Keywords: design procedure, seismic retrofit, existing buildings, braced frame, buckling restrained brace.

1. Introduction

The major seismic events occurred in different areas of the world have led to a growing concern about structural safety. Particularly in the Mediterranean area, the awareness of the detrimental consequences caused by recent seismic events has increased the urgency of enhancing the resiliency of the built environment. Indeed, about 40% of the building stock in Europe has been built before the 1960s and has already overcome the nominal structural life (50 years) [1]. More importantly, the majority of the existing r.c. buildings are inherently vulnerable to seismic actions, since they were generally designed for gravity loads only, or according to old seismic regulations that did not include provisions to promote a ductile response of the structures. An aggravating factor is that the updated seismic zonation has extended the hazard level and today existing structures are often required to sustain seismic forces larger than those considered for their design. Hence, the reduction of the seismic vulnerability of European buildings is crucial to prevent structural collapses and destruction of historical heritage, avoid human casualties and reduce economic losses. Furthermore, because of the growing attention to sustainability, the seismic vulnerability of existing buildings is relevant also from an environmental point of view. In fact, the existing building stock has a deep impact on the environment, since it is responsible for 40% energy consumption and 36% CO_2 emissions in Europe [2]. Considering the waste production, it was observed that the EU construction and demolition waste is about 33% of the total amount of waste. This means that the collapse or the major restoration of buildings after a seismic event has a great impact on the environment in terms of waste production and CO_2 emissions. Moreover, neglecting the seismic risk may lead to incorrect expectations on the actual effect of energy saving measures, because structural vulnerability of existing structures can jeopardize the benefits provided by the energy retrofit interventions [3].

Based on these premises, the selection of the retrofit intervention is a complex task that involves multiple choices and aspects. In fact, it is unlikely to find that a specific retrofit intervention is suitable for every structure and usually the evaluation of the cost/performance ratio is the key to determine the most appropriate intervention. For this reason, recent researches have proposed a holistic approach in a Life Cycle Perspective. It broadens the concept of sustainability, generally related to the sole environmental issues, to include structural safety and seismic vulnerability [4]. The goal of such approach is to solve contemporarily the building deficiencies related to seismic performance, architectural issues, energy efficiency and living discomfort within a single refurbishment solution [5]. Among the available strategies for the seismic rehabilitation of existing buildings, a promising intervention particularly suitable for a holistic approach consists of adding an exoskeleton structure [6]. This is a self-supported structure, set outside and connected to the main structure, and conceived as a "sacrificial" appendage in charge of absorbing seismic actions [7]. The exoskeleton structure shares the typical qualities of other external control systems, for instance rocking wall systems [8]. These strategies (i) operate outside the building and reduce to a minimum the service downtime and evacuation of residents and (ii) limit the strengthening of structural members to those locally interested by the connections to the external frame. In addition to these features, the exoskeleton structure can enhance the economic and ecological efficiency of the retrofitting intervention, offers different architectural solutions and may be used to expand the building spaces [9]. The exoskeleton structures can use wall or shell solutions [4, 10]. In the first case, plane frames are disposed in adhesion or perpendicularly to the facade and the additional stiffness and strength are lumped into few new elements. This approach was followed by Feroldi et al [11] and Lops et al [12], who developed a "double skin facade" for the valorization of existing buildings, while Scuderi applied it for the integrated retrofit of social housing [13]. Similarly, Takeuchi [14] proposed the concept of "Integrated Facade" which treats simultaneously structural retrofit, facade design and environmental design and applied it for the retrofit of the Tokyo Institute of Technology. In the alternative configuration, structural shells encapsulate the original building and can be continuous or discrete. Diagrid or gridshells have been widely used and studied, in particular for high rise buildings [15, 16].

The present paper proposes the seismic upgrade of r.c. existing buildings by steel exoskeleton equipped with BRBs. Indeed, it has been already demonstrated that the insertion of BRBs can increase both the lateral stiffness and the shear strength of the r.c. structure and it is a valid solution for seismic upgrading [17]. In fact, BRBs can modify the distribution of the shear strength along the height in order to promote a widespread yielding of the structure and a more favorable collapse mechanism. Moreover, they can modify the distribution of the lateral stiffness along the height of the building so that the displacement demand can better match the displacement capacity of the structure. However, one of the main drawbacks of this retrofit solution is that BRBs transfer on the existing r.c. columns additional axial forces, which reduce the displacement capacity of columns. Moreover, the insertion of BRBs within the r.c. frame implies the interruption of the building activities and the likely, though temporary, relocation of the occupants. Based on these premises, the insertion of BRBs within a steel exoskeleton is deemed to be a promising retrofit solution, that is able to integrate the strength points of the two technologies to overcome the respective weak points.

The goal of this paper is to extend a design method proposed by the authors for the seismic upgrade of r.c. buildings with BRBs [17] to size a steel exoskeleton endowed with BRBs. The design method aims at obtaining a widespread distribution of the plastic deformations along the height of the building and making the seismic demand compatible with the capacity. To this end, the design procedure permits to define at each story the stiffness and the strength that has to be provided BRBs by choosing appropriate values of cross-section area and yield stress of steel. Columns of the steel external frame are designed to remain elastic and their cross-sections are determined so that the drift demand caused by the axial deformation of steel columns is controlled below a limit value. The parameters that control the design are the design story drift Δ_d, the design ductility μ_d of BRBs and $\Delta_{c,d}$ defined as the permitted rate of Δ_d due to axial elongation of the steel columns.

The proposed design method is first presented and then applied to retrofit two case study r.c. frames representative of buildings with different levels of seismic deficiency. The seismic response of both the bare and the upgraded r.c. frame is evaluated by nonlinear dynamic analysis and represented in terms of the distribution along the height of the drift demand to capacity ratio. From the obtained results, the influence of the design parameters is investigated.

2. The proposed design method

The proposed design method is ruled by two main requirements that have to be fulfilled for an assumed seismic excitation level: (i) the story drift demand Δ has to be not larger than a design story drift Δ_d, (ii) the BRB ductility demand μ corresponding to the story drift capacity has to be not larger than the design BRB ductility μ_d. The design procedure requires two main steps: the first one evaluates whether the original structure needs additional lateral stiffness to sustain seismic action or not. In the second step, the structural members of the steel exoskeleton frame, i.e. BRBs, columns and beams, are sized. All the steps of the design procedure are represented in Fig. 1.

2.1 Definition of the retrofit intervention

To evaluate if the original r.c. frame requires to be upgraded, the design drift capacity Δ_d and the drift demand Δ are evaluated. The design story drift Δ_d is defined as a percentage of the story drift corresponding to the target limit state Δ_{LS}. The value of Δ_{LS} depends on the characteristics of the r.c. frame (mechanical properties of materials, size and detailing of members, etc.), but also on the axial force ratio of the columns, and it is determined by a pushover analysis. In particular, at each story, the drift capacity is evaluated at each step of the pushover analysis as the column chord rotation capacity [18] times the interstory height and is compared to the drift demand corresponding to the current step. When the drift demand at one story equals the capacity, the pushover analysis is stopped and the story drift capacities at the current step are assumed as Δ_{LS}. At the same time, the story lateral strength V_{Rd} is determined by pushover analysis at the attainment of the target limit state and is evaluated as the summation of the lateral strength of the bare r.c. frame and the lateral strength provided by the exoskeleton. At the first iteration, the lateral strength is provided only by the existing frame. The evaluation of the drift demand Δ is based on the equal displacement rule. Particularly,

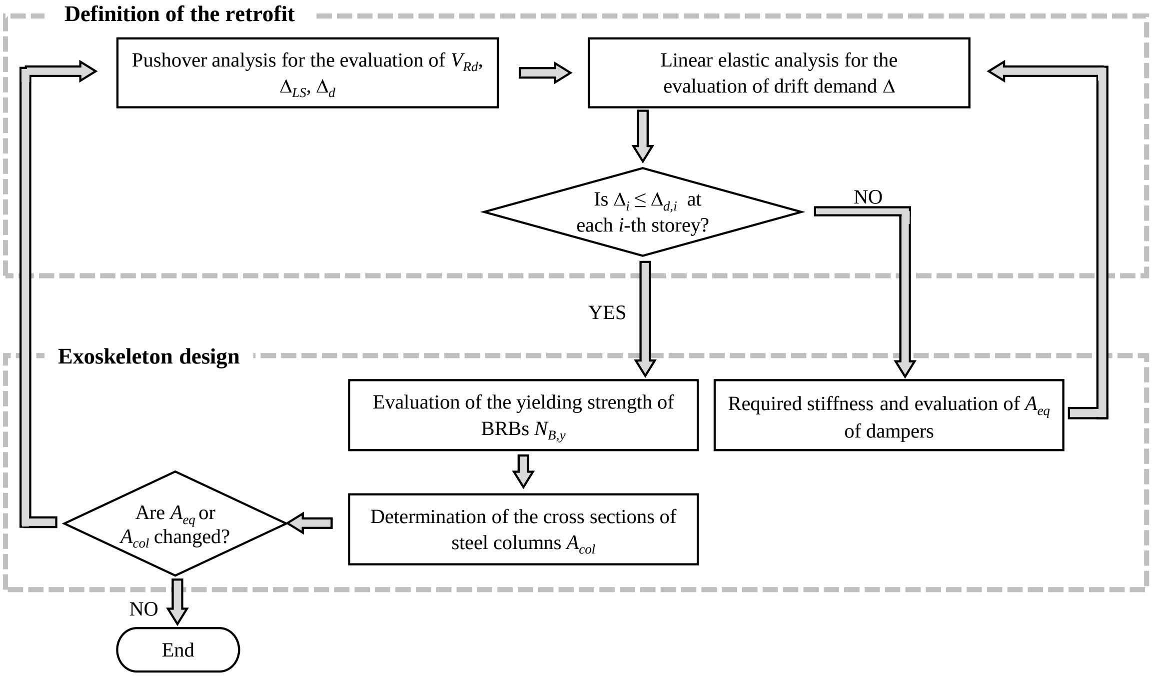
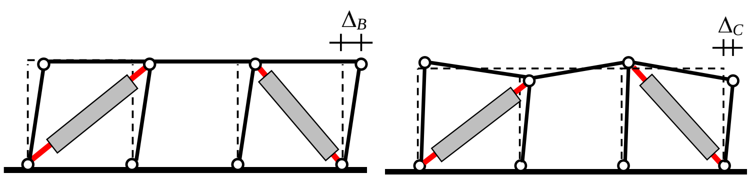


Fig. 1 – Flowchart of the design procedure

the drift Δ_{el} is first determined at each story by an elastic analysis with PGA corresponding to the assumed seismic excitation level. Afterwards, Δ_{el} is multiplied by the coefficient C_R [19] to take into account that the equal displacement rule does not apply for structures whose natural period T_1 is smaller than the corner period T_C of the pseudo-acceleration spectrum. Further, the drift Δ_{el} is modified to take into account that the contribution to the horizontal drift given by the axial deformation of steel columns is overestimated. In fact, during the ground motion BRBs yield for a force level lower than that determined by the elastic analysis. For this reason, the axial force of steel columns, as well as their axial deformation, are overestimated by the elastic analysis. Since the two frames are connected by a rigid diaphragm, all nodes belonging to the same story experience the same horizontal displacement. The story drift Δ_{el}, which is the same for both the r.c. frame and steel exoskeleton, is the sum of the drift Δ_B due to axial deformation of BRBs and the drift Δ_C due to axial deformation of steel columns (Fig. 2). The contribution Δ_C can be calculated from the axial force of steel columns and from kinematic considerations. Thus, the actual drift demand Δ can be obtained by subtracting the contribution in excess of Δ_C to the drift Δ_{el} modified by C_R:

$$\Delta = C_R \cdot \Delta_{el} - \Delta_C \left(1 - \frac{1}{R}\right) \tag{1}$$

where R is the force reduction factor given at each story by the ratio of the elastic shear force V_{el} (i.e. the story shear of the frame from elastic analysis) to the yield lateral strength V_{Rd} (i.e. the story shear of the frame corresponding to the target limit state, which is evaluated by pushover analysis). If the drift demand Δ is larger than the design story drift Δ_d, the insertion of BRBs is needed to provide the frame with the required



(a) (b)

Fig. 2 Contribution to the story drift due to (a) BRBs deformation and (b) column axial deformation stiffness or, if the BRBs are already present, their size has to be increased.

2.2 Exoskeleton design

The BRBs have to provide the lacking stiffness K_T, evaluated as the difference between the required stiffness K_{Req} and the stiffness K_{BF} already provided by the bare r.c. frame. The required stiffness K_{Req} is calculated by the elastic analysis of the frame, and it is equal to the total story shear force over the design story drift Δ_d. The stiffness K_{BF} of the bare frame is calculated as the ratio of the summation of the shear forces carried by columns of the story to the story drift Δ_{el}. Since BRBs and steel columns are assumed to work in series, the drift Δ_B due to the axial deformation of BRBs is calculated as follows:

$$\Delta_B = \frac{V_B}{K_T} - \frac{\Delta_C}{R} \tag{2}$$

where V_B is the shear story carried by BRBs. The lateral stiffness K_B that BRBs have to provide to satisfy the drift requirement is evaluated as the ratio of V_B over the drift Δ_B. Hence, the equivalent cross-section area of the BRBs $A_{B,eq}$ is determined as follows:

$$A_{B,eq} = \frac{1}{n_B} \frac{K_B\, L_B}{E_s \cos^2 \alpha} \tag{3}$$

where L_B is the length of BRBs, n_B is the number of BRBs in each story, E_s is Young's modulus of steel and α is the angle of inclination of the BRBs with respect to the horizontal axis. Since the insertion of the BRBs increases the frame stiffness and modifies the vibration periods and the seismic response of the frame, the design procedure is iterative and ends when the drift requirement ($\Delta \leq \Delta_d$) is satisfied at every story.

Once the equivalent area has been determined, the yield strength $N_{B,y}$ of BRBs has to be designed. The yield strength of BRBs is determined to prevent the exceedance of the ductility capacity of BRBs. The ductility demand of BRBs is evaluated as the ratio of the drift $\Delta_{B,max}$ due to the sole deformation of BRBs at the target limit state over the drift $\Delta_{B,y}$ due to the axial elongation of BRBs at yielding ($\Delta l_{B,y}$). The value of $\Delta_{B,max}$ is determined by subtracting the rate of drift due to the axial deformation of columns from the total drift Δ (calculated by Eq. (1)), according to the following equation:

$$\Delta_{B,max} = \left(\Delta - \frac{\Delta_C}{R} \right) \cdot \frac{\Delta_{SL}}{\Delta_d} \tag{4}$$

The drift $\Delta_{B,y}$ can be related to the yield strength $N_{B,y}$:

$$\Delta_{B,y} = \frac{\Delta l_{B,y}}{\cos \alpha} = \frac{N_{B,y}\, L_B}{E_s\, A_{B,eq} \cos \alpha} \tag{5}$$

Equating the BRBs ductility demand ($\Delta_{B,max}/\Delta_{B,y}$) to the design ductility μ_d, the yielding strength of BRBs can be evaluated as:

$$N_{B,y} = \frac{E_s\, A_{B,eq}\, \Delta_{B,max} \cos \alpha}{\mu_d\, L_B} \tag{6}$$

where μ_d is calculated reducing the ductility capacity of BRBs μ_{LS} by a safety factor γ_μ. Eventually, the cross-section of the steel beams and columns are designed to remain elastic. Considering the axial force transmitted to columns by yielded and fully hardened BRBs, the size of the column cross-sections is determined at each story to satisfy two conditions: (i) columns have to sustain the axial force induced by BRBs, in compliance with the capacity design criteria for steel braced structures [18]; (ii) columns must be stiff enough to ensure that at the top story, the drift caused by the cumulative axial deformation of columns of all the stories is not larger than apercentage ρ_c of the design story drift of the top story $\Delta_{d,top}$. Hence, at each story, the upper limit to the story drift $\Delta_{c,d}$ caused by the axial deformation of columns depends on the number of stories n as follows:

$$\Delta_{c,d} = \rho_c \cdot \Delta_{d,top} \cdot [2(n-1)+1] \tag{7}$$

. Since the beam of the exoskeleton are supposed to sustain only their own weight, their cross-section is assumed equal to IPE 180. Once the steel members are determined, the pushover analysis is run to estimate the seismic stiffness and strength of the upgraded structure.

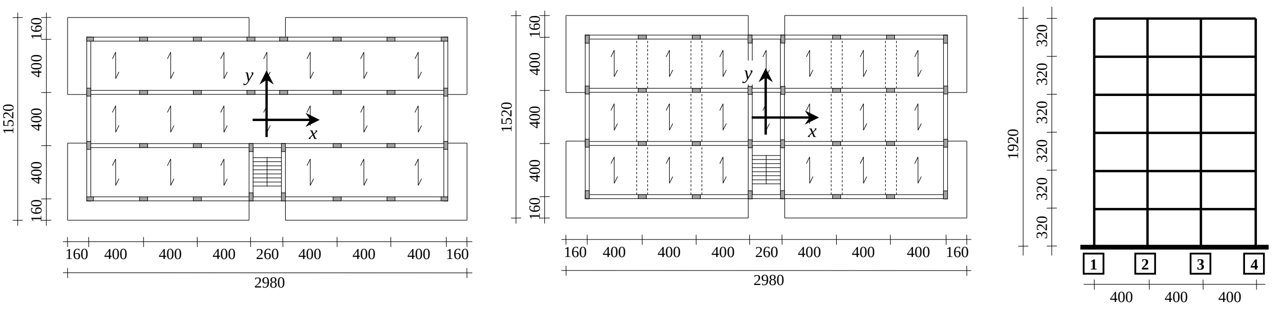
3. The case studies

Two r.c. framed buildings were designed as case studies: the first one aimed to represent Italian buildings designed to sustain gravity loads only, while the second exemplified the existing buildings designed according to old Italian seismic standards for low seismicity areas. The plan layouts of the designed structures are shown in Fig. 3. In both cases, the buildings are six-story high and have four frames with seven bays along the *x*-direction. With regards to the *y*-direction, the building designed for gravity loads (Fig. 3a) exhibits four frames: the two outermost frames have three bays, the other two next to the staircase have only one bay. Instead, in the building designed according to old seismic regulations (Fig. 3b), eight three-bay frames are arranged along the *y*-direction. The GL frame is drawn from the building designed for gravity loads, and the SR frame is drawn from the building designed for low seismicity areas; both are the outermost frames laying along the *y*-axis. These frames have the same geometrical scheme and their features are shown in Fig. 3c.

The cross-sections of beams and columns of the GL frame were designed according to the Italian regulations in force during the seventies [20 - 22] and the obtained dimensions are listed in Table 1. Dead and live gravity loads are determined considering the nominal values given in [23]. The design internal forces of the structural members are determined considering only gravity loads. Concrete cross-section and steel reinforcement of beams and columns are determined by the allowable stress method [22]. For beams, the minimum reinforcement ratio of the tension zone prescribed in [22] is equal to 0.0015. Columns are designed to resist compressive axial force, while the bending moment is neglected. The design axial force of the column N is evaluated according to the tributary area concept and the minimum required cross-sectional area of the column $A_{c,req}$ is calculated as follows:

$$A_{c,req} = \frac{N}{0.7\, \bar{\sigma}_c \,(1 + n\, \rho_l)} \tag{8}$$

where $\bar{\sigma}_c$ is the allowable stress of concrete, n is the homogenization coefficient for steel rebars assumed equal to 10, and ρ_l is the ratio of the longitudinal rebar area A_s to $A_{c,req}$ assumed equal to the minimum value required by the code (0.006). The characteristic compressive cubic strength R_{ck} of concrete is assumed equal to 25 MPa (corresponding to f_{ck} equal to 20 MPa); steel grade Feb38K with characteristic yield strength



(a) (b) (c)

Fig. 3 – (a) Plan view of the GL buildings; (b) Plan view of the SR building; (c) Analyzed frame

Table 1 – Cross-section of columns and beams

Story	GL frame			SR frame		
	Columns		Beams	Columns		Beams
	1 and 4	2 and 3		1 and 4	2 and 3	
6th	30x30	30x30	30x60	30x60	30x60	30x50
5th	30x30	30x30	30x60	30x60	30x60	30x50
4th	30x30	30x30	30x60	30x60	30x60	30x50
3rd	35x30	30x40	30x60	30x60	30x60	30x60
2nd	40x30	30x50	30x60	30x60	30x60	30x60
1st	50x30	30x60	30x60	30x60	30x60	30x60

f_{yk} = 375 MPa is employed for rebars. These design assumptions lead to allowable stresses for concrete and rebars equal to 8.5 and 215 MPa, respectively. The minimum rebar area of columns A_s has to be not smaller than 0.3% of the actual cross-sectional area A_c of the column and 0.6% $A_{c,req}$. Spacing of stirrups is 150 mm for columns and 200 mm for beams.

The structural elements of the SR frame are sized considering also the effect of seismic forces. The lateral force method of analysis is applied. The total design seismic force F_h is calculated as the product of the seismic coefficient C (depending on the seismicity of the site), the response coefficient R (ordinate of the design acceleration spectrum normalized with respect to g) and the total seismic weight of the building W, as prescribed by the seismic code [23] for residential buildings with r.c. structure. Assuming a low seismicity site, the seismic coefficient was set equal to 0.04 and the response coefficient R is assumed unitary, as suggested by the old Italian seismic code. The floor seismic weight is equal to 3515 kN at all floors and the total design seismic force F_h is 843.9 kN. The analyzed frame has the same geometrical scheme of the GL frame (Fig. 3c). However, it was designed to sustain one fourth of the total seismic force, since the contribution to lateral strength and stiffness provided by the internal frames with flat beams is negligible. The design internal forces of beams and columns are evaluated considering the most unfavorable combination of the gravity loads and seismic forces. The sizes of cross-sections and rebars are determined according to the allowable stress method stipulated in [21]. However, the cross-sections of columns (listed in Table 1) are selected not smaller than those of beams, to avoid excessive concentration of damage in one story. In this regard, the same concrete assumed for the GL frame is adopted for beams and columns. Instead, steel grade Feb44k with a characteristic yield strength f_{yk} = 430 MPa is used for reinforcement bars. A more detailed description of both the case studies may be found in [17].

4. Seismic assessment of case study frames and design of the seismic upgrading

To investigate the need for seismic upgrading, the seismic performance of the two case study frames has been assessed and the seismic demand was compared to the corresponding capacity of the frame. To this end, a two-dimensional numerical model was built in OpenSEES [24] to run nonlinear Incremental Dynamic Analyses (IDA). Hence, the upgrading intervention is sized by the proposed procedure.

4.1 Numerical model

In the numerical model the rigid diaphragm effect of the concrete deck is simulated by constraining all the nodes belonging to the same floor to have the same horizontal displacement. Masses are lumped at the floor levels and P-Δ effects are considered by adding a leaning column that is loaded by vertical forces equal to the floor weights. The "Beam With Hinges Element" implemented in OpenSEES is used for columns and beams to simulate elastic members with plastic hinges at their ends. The length of the plastic hinge is assumed equal to the depth of the cross-section, and a fiber cross-section including both concrete and steel components is assigned to each plastic hinge. The Mander constitutive law (*Concrete 04* in OpenSees) is assigned to concrete fibers. An elasto-plastic material with strain kinematic hardening constitutive law (*Steel 02* in OpenSees) is assigned to steel fibers. The parameters used for materials are summarized in Table 2. Note that, the SR frame was supposed to be realized with concrete and steel with compressive strength and yielding strength lower than those adopted for the design. The area, the moment of inertia of

Table 2 – Characteristics of materials for the nonlinear dynamic analysis

Concrete	GL	SR
Cylinder compressive strength (MPa)	29	20
Young's modulus (MPa)	30280	27085
Strain at maximum strength	2×10^{-3}	2×10^{-3}
Tensile strength in tension (MPa)	0.00	0.00

Rebars	GL and SR
Yielding strength (MPa)	375
Young's modulus (MPa)	210000
Ultimate strain in tension	7.5×10^{-5}
Strain-hardening ratio	0.0049

concrete cross-section and the Young's modulus of concrete are assigned to the elastic element. Furthermore, a "ZeroLength Element" characterized by a small axial stiffness and a large shear and flexural stiffness is added at one end of each beam. This element acts as an axial release and prevents the development of axial force in beams [25].

4.2 Seismic performance of case studies

The performance of the bare frames is evaluated by means of IDA. The numerical model is subjected to a set of ten artificial ground motions, compatible with the EC8 elastic spectrum for soil type C and characterized by 5% damping ratio. The SIMQKE computer program [27] is used to generate the ground motions. Each ground motion is characterized by a total duration of 30.5 s and is enveloped by a three-branch compound function. The duration of the strong motion phase of the accelerogram is equal to 7.0 s and this choice follows previous investigations [28]. To run IDA, the reference set of ground motions is scaled to PGA ranging from 0.05 g to 0.60 g, in step of 0.05 g. The seismic performance of the bare frames is evaluated in terms of maximum story drift demand (Δ/H, being H the interstory height) and maximum story drift demand to capacity ratio (Δ/Δ_{LS}). The displacement capacity Δ_{LS} is evaluated at each step of the analysis and is referred to the Near Collapse (NC) limit state. Given the seismic excitation level, each response parameter is evaluated for the ten accelerograms. Then, the mean over the ten ground motions is determined and the distribution along the height of the mean values of the response parameters is shown in Fig. 4

The GL frame showed a damage concentration at the fourth story (Fig. 4 (a)), where the drift demand overcame the capacity (Δ/Δ_{LS}>1) for a PGA equal to 0.20 g (Fig. 4 (b)). Furthermore, for PGAs larger than 0.20 g, several numerical instabilities occurred, which can be related to the collapse of the structure. In particular, the GL frame could not overcome PGAs larger than 0.25 g. The SR frame showed an almost uniform distribution of the story drift demand along the height (Fig. 4 (c)), with a slight concentration of

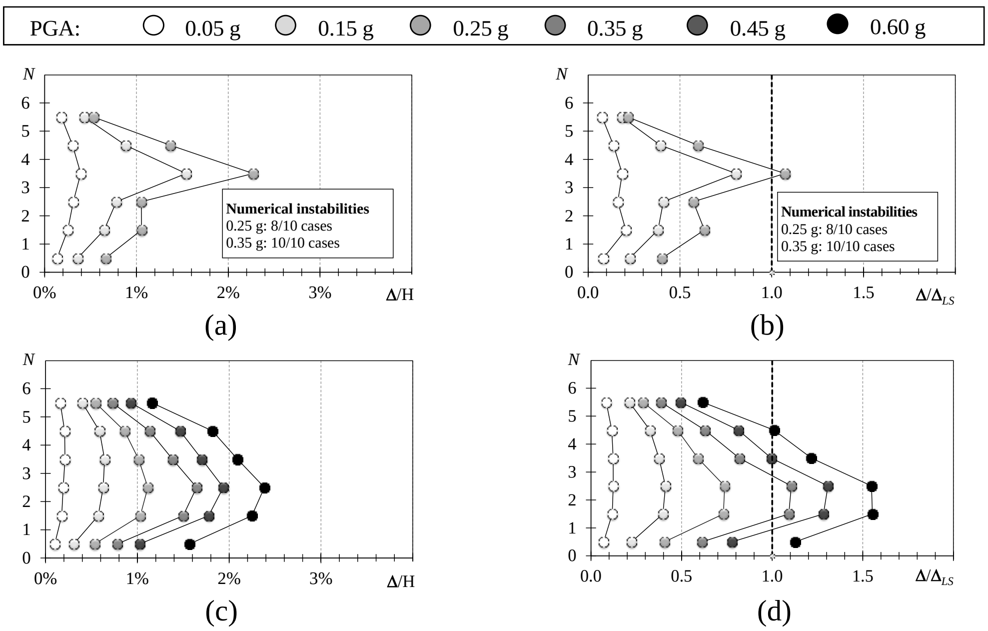


Fig. 4 – Heightwise distribution of story drift angle and ratio of drift demand to capacity of frames (a, b) GL and (c, d) SR

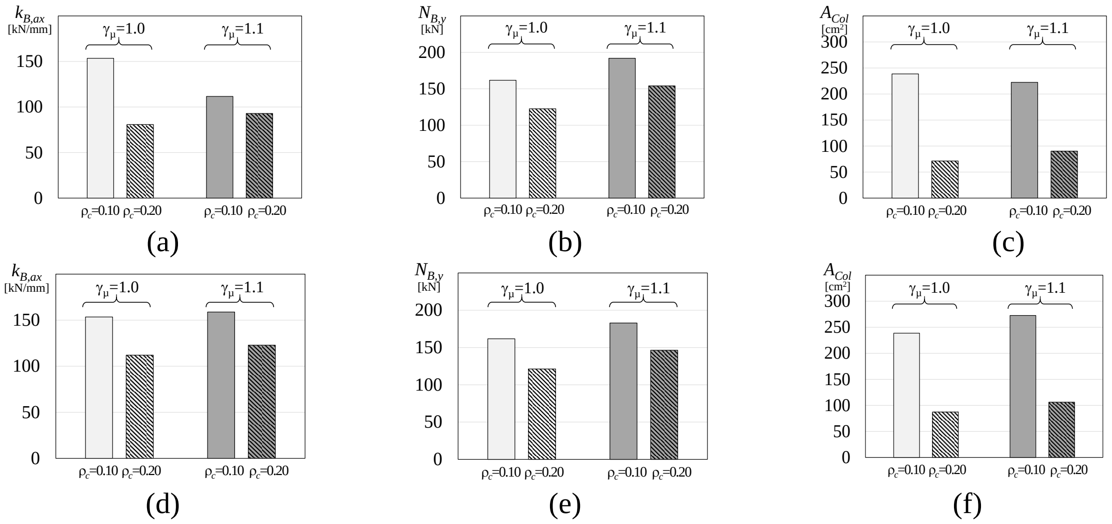


Fig. 5 – Comparison of the average values of (a, d) BRB axial stiffness, (b, e) BRB yielding strength and (c, f) area of steel columns for the seismic upgrading of (a, b, c) GL frame and (d, e, f) SR frame.

damage at the third story for very large PGAs (from 0.45 g on). However, the drift demand exceeded the capacity for a PGA close to 0.25 g (Fig. 4 (d)). Since the minimum PGA corresponding to the attainment of the NC limit state stipulated by EC8 is 0.60 g, none of the case study frames met the code requirement. Hence, both the frames needed to be seismically upgraded.

4.3 Design of seismic upgrading

The two case study frames are retrofitted by adding a steel exoskeleton frame equipped with BRBs in the two lateral spans. The axial stiffness and yield strength of BRBs and the cross-section of steel columns are determined by the proposed design method, assuming the NC limit state as target. The numerical model adopted for the elastic analyses and the pushover analyses required by the design procedure is the same described in Section 4.1, with the only exception that *P*-Δ effects are neglected for the design. Particularly, beams and columns of the exoskeleton are modelled as "Beam With Hinges Element", with cross-sections discretized into fibers, in consistency with the members of the r.c. frame. Steel is modelled as elasto-plastic material with strain kinematic hardening and yielding strength equal to 235 MPa. BRBs are included in the numerical model as truss elements with the cross-sectional area equal to the designed equivalent area $A_{B,eq}$. The cyclic behavior is simulated by the material model proposed by Zona and Dall'Asta [26] for steel buckling restrained braces. The stiffness properties of this model are defined by the initial elastic stiffness k_0, assumed equal to the Young modulus, and the post-yield stiffness k_1, evaluated as the product of the kinematic strain hardening ratio k_h times k_0. The strength of the material is defined by the yield stress $f_{y,eq}$, and the maximum yield stress in tension $f_{y,max}$ and in compression $f_{y,min}$ for the fully saturated isotropic hardening condition [17].

The design of the seismic upgrading of each r.c. frame is conducted four times. The design story drift is fixed equal to $0.6\Delta_{LS}$ [17], while the column drift contribution $\Delta_{c,d}$ is calculated by Eq. (7) assuming ρ_c equal to 0.10 or 0.20 of $\Delta_{d,top}$ and the ductility safety factor γ_μ is taken equal to 1.0 or 1.1. The results of the design are shown for GL and SR frame in Fig. 5 (a-c) and (d-f), respectively, in terms of axial stiffness and yield strength of BRBs, and average area of the cross-section of steel columns. For the sake of simplicity, each column shows the average value along the height of the aforementioned parameters obtained by the four designs. Larger values of ρ_c lead to BRBs with smaller cross-section area and lower yielding strength, and reduce the cross-sections of columns as well. For a fixed value of ρ_c, the yielding strength of BRBs and the cross-sections of columns and BRBs generally increase with the ductility safety factor γ_μ. However, the ductility safety factor γ_μ does not influence the design as much as the column drift contribution ρ_c.

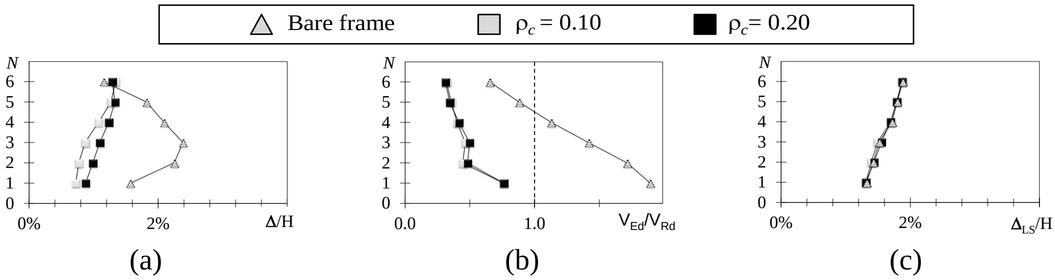


Fig. 6 – Seismic response at NC limit state of SR frame upgraded by $\Delta_d = 0.60$, $\gamma_\mu = 1.1$ and $\rho_c = 0.10$ or 0.20

5. Seismic performance of the upgraded case studies

The seismic response of the upgraded frames was evaluated with reference to the Significant Damage (SD) and Near Collapse (NC) limit state. Nonlinear dynamic analysis is run using the same numerical model described in section 4.1 and 4.3. The set of accelerograms already presented in section 4.2 is scaled to PGA equal to 0.35 g and 0.60 g, corresponding to a probability of exceedance of 10% (SD) and 2% (NC) in 50 years, respectively.

With reference to the NC limit state, Fig. 6 a, b and c show for the SR frame the distribution along the height of drift demand, drift demand to capacity ratio and shear demand to capacity ratio, respectively. The results of the bare frame (grey triangles) are plotted alongside those of the frame retrofitted considering $\Delta_d = 0.60$, γ_μ=1.1 and $\rho_c = 0.10$ (grey squares) or $\rho_c = 0.20$ (black squares). Regardless of the value assigned to ρ_c, the addition of the steel exoskeleton with BRBs drastically reduces the story drift demand compared to the bare frame, and promotes a more uniform distribution of story drift along the height of the building (Fig. 6 a). The exoskeleton is also able to reduce the shear demand well below the capacity at each story (Fig. 6 b) and prevents the collapse of the frame caused by shear failure of r.c. columns. Furthermore, since BRBs are included in the external exoskeleton, they do not transmit any additional axial force to r.c. columns and the displacement capacity of the bare frame is not reduced (Fig. 6 c). Even if not shown in figure, the same trend was found also for the SR frame retrofitted with γ_μ=1.0 and for GL frame considering all the above-mentioned values of γ_μ and ρ_c.

The values assigned to ρ_c and γ_μ have a more significant impact on the verification of demand to capacity ratio for r.c. columns (Δ/Δ_{LS}) and ductility verification of BRBs (μ/μ_{LS}), as shown in Fig. 7 for GL

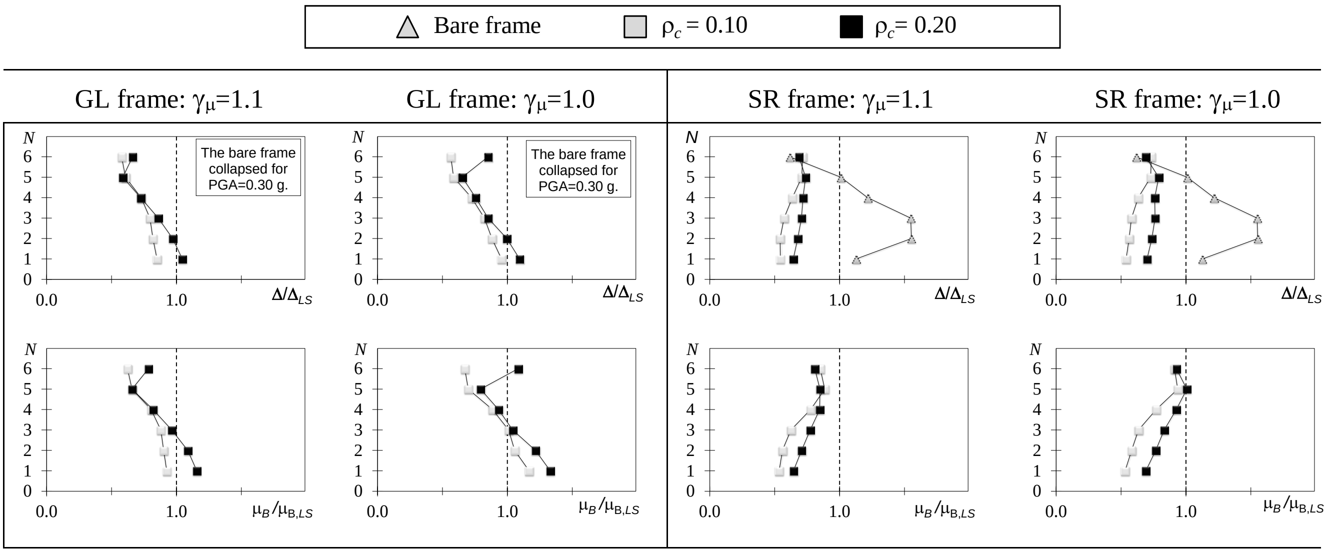


Fig. 7 – Seismic response at NC limit state of GL frame (left columns) and SR frame (right columns) upgraded by γ_μ=1.1 and γ_μ=1.0.

frame and SR frame. For the sake of brevity, all the figures refer to the NC limit state. In the case of the SR frame, all the combinations of the design parameters γ_μ and ρ_c allow the rigorous fulfillment of the drift verification of r.c. columns and basically also the ductility verification of BRBs. The seismic deficiency of the GL frame is more significant and more restrictive values need to be assigned to γ_μ and ρ_c. In the case γ_μ is assumed equal to 1.0, none of the verifications (Δ/Δ_{LS} or μ/μ_{LS}) is satisfied for the GL frame. In the case γ_μ is equal to 1.1, the upgraded GL frame virtually satisfies the verification of r.c. columns for both values of ρ_c. On the contrary, the verification of BRBs of the GL frame is satisfied only if ρ_c is 0.10. In fact, a lower value of ρ_c leads to larger cross-sections of BRBs and steel columns that in turn increase the lateral stiffness and reduce both the ductility demand of BRBs and the story drift demand. Based on these results, the combination of values of the design parameters that allows the fulfillment of all the verifications for NC limit state for both the considered frames is ρ_c = 0.10 and γ_μ=1.1. However, it should be observed that the design of the seismic upgrading intervention by ρ_c = 0.10 leads to a very heavy steel exoskeleton, especially for the size of the columns. A more cost-effective solution is obtained assuming ρ_c = 0.20 and γ_μ=1.1. These values lead to the verification of the SD limit state for both the SR and GL frames (even if not shown in figure) and the verification of the NC limit state for the SR frame.

6. Conclusions

This paper proposes a design method for seismic upgrading of existing RC frame by means of a steel exoskeleton endowed with BRBs. The design procedure, following a displacement-based approach, is ruled by the design story drift Δ_d, the percentage column drift contribution ρ_c and the ductility safety factor γ_μ. Two case study frames were designed to be representative of existing r.c. frames suffering from different levels of seismic deficiencies. Incremental nonlinear dynamic analysis showed that both frames needed to be upgraded to satisfy the limit states prescribed by EC8. Hence, the proposed method was applied to design the seismic retrofit of the case study frames. In particular, the design procedure was repeated different times to investigate the influence of the design parameters. It was found that the dimensions of both BRBs and steel columns significantly decrease with larger values of ρ_c. Instead, larger values the ductility safety factor γ_μ generally increase the size of both BRBs and steel columns, even though the influence of γ_μ on the design results is less significant than that due to ρ_c. To test the effectiveness of the seismic retrofit intervention, the seismic response of the upgraded frames was determined by nonlinear dynamic analyses considering ground motions with probability of exceedance of 10% and 2% in 50 years. The introduction of the exoskeleton with BRBs reduced the maximum drift demand for both the case study frames and promoted a more uniform distribution of drift along the height. Furthermore, it prevented the shear collapse of RC columns. In particular, even though the target of the design was the NC limit state, the design method applied with γ_μ=1.1 and ρ_c=0.10 allowed the fulfillment of both the NC and SD limit states. Note that, the verification of the SD limit state is satisfied even assuming γ_μ=1.1 and ρ_c=0.20.

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