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# A DESIGN PROCEDURE FOR SEISMIC UPGRADING OF EXISTING RC FRAMES BY BRBS

F. Barbagallo[1], A. Ghersi[2], E.M. Marino[3], P.R. Stramondo[4]

[1] *Ph.D. student, University of Catania, fbarbaga@dica.unict.it*
[2] *Professor, University of Catania, aghersi@dica.unict.it*
[3] *Assistant professor, University of Catania, emarino@dica.unict.it*
[4] *Doctor researcher, University of Catania, pstramon@dica.unict.it*

## *Abstract*

In the world, many existing buildings with RC framed structure were designed according to old seismic standards and, consequently, present structural deficiencies. Embedding Buckling Restrained Braces (BRBs) in the RC framed structure is a promising technique for seismic upgrading of existing buildings. In fact, BRBs, if properly sized, can improve many features of the seismic response of existing buildings. BRBs provide the RC frame with additional dissipation capacity, which reduces drifts. Furthermore, BRBs can avoid drift concentration at few stories, thus promoting a favorable and dissipative collapse mechanism. Finally, the heightwise distribution of the size of BRBs can be optimized, so that drift demand is tuned at every story with the drift capacity of the RC frame. In this paper, a design method for seismic upgrading of existing RC frames by BRBs is presented. According to this method, BRBs are designed to fulfill stiffness and strength requirements in order to achieve the target limit state. The parameters that rule the method are the design story drift $\Delta_d$, and the behavior factor $q$.

As an example, the proposed design method is applied to retrofit a RC frame which does not satisfy the minimum requirements stipulated by Eurocode 8 for RC framed structures in occurrence of strong ground motions. The case study frame is designed according to the provisions stipulated by the old Italian seismic code for structures located in low seismicity zones. The design procedure is applied considering different pairs of values of $\Delta_d$ and $q$, in order to investigate the influence of the parameters that control the outcome of the design on the seismic performance of the upgraded frame. The seismic response of the upgraded frame is evaluated by nonlinear dynamic analysis and its seismic performance is compared to that of the bare RC frame. The results shows the effectiveness of the design method and provide information on properly setting of the design parameters $\Delta_d$ and $q$.

*Keywords: seismic retrofitting, RC frames, existing buildings, hysteretic dampers, BRB*

# 1. Introduction

Despite many regions of the world are extremely prone to seismic activity, a large number of buildings spread on such territories are not able to survive strong ground motions. In fact, many of the existing RC buildings were designed according to old seismic regulations, which did not included provisions to promote a ductile response of the structures. Furthermore, despite some structures were designed to sustain seismic forces, the considered seismic level may results significantly lower than that imposed today by the update seismic zonation. Because of this, all these structures suffer from sever seismic deficiencies, such as inadequate lateral strength and stiffness, low dissipative capacity and concentration of damage. Thus, they are vulnerable to horizontal actions and the seismic retrofitting of such structures is required. To this end, the introduction of the Buckling Restrained Braces (BRBs) can be considered as a promising approach. In fact, the insertion of BRBs can increase to proper value both the lateral stiffness and the shear strength of the structure. BRBs can modify the distribution of the shear strength along the height so as to promote a widespread yielding of the structure and therefore a more favorable collapse mechanism during strong ground motions. Moreover, they can modify the distribution of the lateral stiffness along the height so that the displacement demanded by the ground motion can better match the displacement capacity of the structure.

The present paper introduces a design method of BRBs for the seismic upgrade of RC existing buildings. The developed 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 stiffness and strength of BRBs by choosing appropriate values of cross-section area, length of the yielding segment and yield stress of the steel. The design method aims at satisfying three requirements: (i) the drift demand must be lower than the drift capacity; (ii) the ductility demand of BRBs must be lower than their ductility capacity; (iii) the total lateral strength, provided by BRBs and RC frame, have to be equal to a certain minimum level. The parameters that control the design are the behavior factor $q$ and the design story drift $\Delta_d$. The first one rules the minimum lateral strength level required to the structure, while the second one, which is assumed as a fraction of the maximum drift that the bare RC frame can accommodate, control the additional stiffness to be provided by BRBs.

In the first part of the paper, the proposed design method is presented. Afterwards, it is applied to retrofit a RC frame representative of buildings designed for an insufficient level of seismic force, considering different combinations of the design parameters. The seismic response of the bare RC frame and the upgraded RC frame is evaluated by means of 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 $q$ and $\Delta_d$ on the demand to capacity ratio of the upgraded frame is investigated.

# 2. The proposed design method

The proposed design method is ruled by three requirements that has to be fulfilled for an assumed seismic excitation level: (i) the story drift demand $\Delta$ has to be not larger than a design story drift $\Delta_d$, (ii) the BRB ductility demand corresponding to the story drift capacity has to be not larger than the BRB ductility capacity $\mu_{B,LS}$, (iii) the total lateral strength $V_{Rd}$ at each story has to be not smaller than the design story shear force $V_{Ed}$. Based on these requirements, the design method develops into two main steps. Step 1 yields to the determination of the stiffness of the BRBs, so that the drift demand of the frame is reduced below its capacity (fulfillment of (i)). Step 2 leads to the evaluation of the yield strength $N_{B,y}$ that has to be assigned to BRBs to not overcome the ductility capacity of BRBs (fulfillment of (ii)) and to promote a uniform distribution along the height of the ratio of lateral strength to seismic shear demand (fulfillment of (iii)). More details may be found in [1].

## 2.1 Step 1: Determination of the stiffness of BRBs

In Step 1 the drift demand $\Delta$ and the design drift capacity $\Delta_d$ are evaluated. The design story drift $\Delta_d$ is defined as a percentage of the story drift corresponding to the target limit state $\Delta_{LS}$. According to seismic codes [2], the evaluation of the story drifts $\Delta_{LS}$ depends on features of RC frame (mechanical properties of materials, size and detailing of members, etc.), but also on axial force of columns.

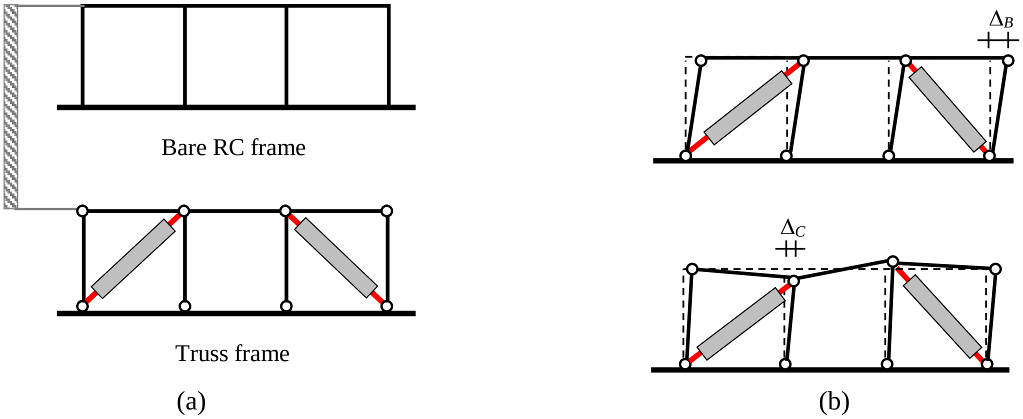



Fig. 1 – (a) Models for the evaluation of the contribution to the stiffness of the RC frame with BRBs (b) Contribution to the story drift due to BRBs deformation (up) and column axial deformation (down)

The drift demand Δ is determined from the value of the drift $\Delta_{el}$, which in turn is provided at each story by an elastic response spectrum analysis with PGA corresponding to the assumed seismic excitation level. However, $\Delta_{el}$ is subjected to two corrections. The first correction aims at evaluating the actual contribution provided by the axial deformation of columns to $\Delta_{el}$. Since during the ground motions BRBs yield for a force level lower than that determined by the elastic analysis, the axial force of columns, as well as their axial deformation, are overestimated by the elastic analysis. The second correction takes into account that the equal displacement rule does not apply for stiff structures whose natural period $T_1$ is smaller than the corner period of the spectrum $T_C$. To evaluate the first correction, the RC frame with BRBs is assumed to behave as two frames in parallel (Fig. 1a): the bare RC frame and the (BRBs) truss frame. The drift $\Delta_{el}$ is equal to the drift of the truss frame, which in turn is the sum of the drift $\Delta_B$ due to axial deformation of BRBs, and the drift $\Delta_C$ due to axial deformation of columns (Fig. 1b). The contribution $\Delta_C$ can be calculated from the axial force of columns and from kinematic considerations. Then, the contribution in excess of $\Delta_C$ can be subtracted from the drift $\Delta_{el}$. Finally, the second correction is applied to the obtained drift demand Δ by multiplying it for the coefficient $C_R$ and the following equations are obtained:

$$\Delta = C_R \left[ \Delta_{el} - \Delta_C \left( 1 - \frac{1}{R_F} \right) \right] \tag{1}$$

$$C_R = \begin{cases} 1 & \text{for} \quad T_1 \geq T_C \\ \dfrac{1}{R_F} \left[ 1 + \left( R_F - 1 \right) \dfrac{T_C}{T_1} \right] & \text{for} \quad T_1 < T_C \end{cases} \tag{2}$$

where $R_F$ is the force reduction factor given by the ratio of the elastic base shear force $V_{el}$ (i.e. the base shear of the frame from elastic analysis) to the yield lateral strength $V_{Rd}$ (i.e. the base shear of the frame corresponding to the target limit state, which is evaluated by pushover analysis).

At each story, drift demand Δ and design drift $\Delta_d$ are compared. If Δ exceeds $\Delta_d$, the insertion of BRBs is required or, if the BRBs are already present, their size has to be increased. In both cases, 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 RC 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 $\Delta_d$. 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 over the story drift $\Delta_{el}$. Since BRBs and columns are assumed to work in series, the drift $\Delta_B$ due to the axial deformation of BRBs is calculated as follows:

$$\Delta_B = \frac{V_B}{K_T} - \frac{\Delta_C}{R_F} \tag{3}$$

where $V_B$ is the shear story carried by BRBs. The stiffness that BRBs have to provide to satisfy the drift requirement is evaluated as:

$$K_B = \frac{V_B}{\Delta_B} \tag{4}$$

Thus, the cross-section area of the BRBs $A_{B,eq}$ is determined as follows:

$$A_{B,eq} = \frac{1}{n_B} \frac{K_B\, L_B}{E_s \cos^2 \alpha} \tag{5}$$

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 $\alpha$ 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 ran iteratively until the drift requirement ($\Delta \leq \Delta_d$) is satisfied at every story.

## 2.2 Step 2: Determination of the strength of BRBs

The second step of the design procedure provides the yield strength $N_{B,y}$ that has to be assigned to BRBs in order to (i) satisfy the ductility requirement and (ii) provide at each story a minimum lateral strength.

### 2.2.1 BRB ductility requirement

The yield strength of BRBs $N_{B,y}$ is firstly 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 deformation of BRBs at the target limit state over the BRBs yielding drift $\Delta_{B,y}$. The value of $\Delta_{B,max}$ is determined by subtracting the rate of drift due to columns from the total drift $\Delta$ (calculated by Eq. (1) at the end of the iterative procedure), according to the following equation:

$$\Delta_{B,max} = \left( \Delta - \frac{\Delta_C}{R_F} \right) \cdot \frac{\Delta_{SL}}{\Delta_d} \tag{6}$$

The drift $\Delta_{B,y}$ depends on the axial elongation of BRB at yielding and is related to the yield strength $N_{B,y}$ as follows:

$$\Delta_{B,y} = \frac{\Delta l_{B,y}}{\cos \alpha} = \frac{N_{B,y}\, L_B}{E_s\, A_{B,eq} \cos \alpha} \tag{7}$$

From the equation of the BRBs ductility demand to the BRBs ductility capacity $\mu_{B,LS}$, the yielding strength of BRBs can be evaluated as:

$$N_{B,y} = \frac{E_s\, A_{B,eq}\, \Delta_{B,max} \cos \alpha}{\mu_{B,LS}\, L_B} \tag{8}$$

### 2.2.2 Lateral strength requirement

In this step the required lateral strength $V_{Ed}$ and available lateral strength $V_{Rd}$ are evaluated. The required lateral strength of the frame $V_{Ed}$ is provided by the linear elastic analysis of the frame performed in Step 1, and it is equal to the elastic story shear force sustained by the frame for the assumed seismic excitation level, reduced by the behavior factor $q$.

The lateral strength $V_{Rd}$ at each story of the upgraded frame is equal to the summation of the available lateral strength of the bare RC frame $V_{BF,Rd}$ and that provided by BRBs $V_{B,Rd}$. The lateral strength of the bare frame is given by the summation of the shear forces in columns and can be determined by a pushover analysis stopped at the attainment of the target limit state. The strength provided by BRBs is calculated as the summation of the horizontal components of the axial forces of the BRBs at the considered story. The additional lateral strength to be provided by BRBs $V_{B,Ed}$ can be determined subtracting $V_{BF,Rd}$ from the story design shear force $V_{Ed}$:

$$V_{B,Ed} = V_{Ed} - V_{BF,Rd} \tag{9}$$

If the ductility of the BRBs $\mu_B$ is related to $\Delta_{B,max}$, the resisting shear force of BRBs corresponding to the target limit state, i.e. when the rate of story drift due to BRB deformation is equal to the $\Delta_{B,max}$, can be written as follows:

$$V_{B,Rd} = n_B \left[ \left(1.15 - k_h\right) N_{B,y} + k_h \frac{E_s\ A_{B,eq}\ \Delta_{B,max} \cos\alpha}{L_B} \right] \cos\alpha \tag{10}$$

Equating Eq. (10) to the additional lateral strength to be provided by BRBs $V_{B,Ed}$, the required yield strength of BRBs can be evaluated as:

$$N_{B,y} = \frac{1}{1.15 - k_h} \left( \frac{V_{B,Ed}}{n_B \cos\alpha} - k_h \frac{E_s\ A_{B,eq}\ \Delta_{B,max} \cos\alpha}{L_B} \right) \tag{11}$$

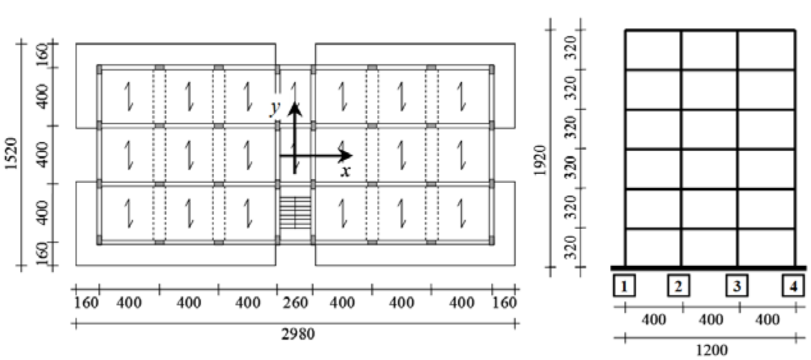
The yield strength $N_{B,y}$ obtained by Eq. (11) is compared to that determined by Eq. (8) according to the ductility requirement and the largest between the two values is adopted for the BRBs of the story. Thus, the BRBs are inserted and the pushover analysis is performed again. The procedure is repeated iteratively until convergence is attained.

## 3. Case study

The developed method has been applied to retrofit a RC frame designed following the provisions stipulated by old Italian codes for low seismicity zone. To evaluate the performance of the rehabilitated frame nonlinear dynamic analyses were run and the results compared to those of the bare frame, as shown in the next Section. To this end, a numerical model of the bare frame and the upgraded frame was built using the software OpenSees [3].

### 3.1 Description of the analyzed frame

The considered frame is drawn from a six-story RC framed structure designed for seismic forces according to the old Italian seismic code [4]-[6] in force in the nineties. The structure plan (Fig. 2a) is symmetric with respect to the $y$-axis and presents four seven-bay frames along $x$-direction and eight three-bay frames along $y$-direction. The effect of seismic force is evaluated by the lateral force method of analysis. The total design seismic force $F_h$ is determined, according to the seismic code [6] for residential buildings with RC structure, as function of the seismic coefficient $C$ (depending on the seismicity of the site), the response coefficient $R$ (ordinate of the design




(a) (b)

Fig. 2 – (a) Plan view of the analyzed building; (b) features of the analyzed RC frame

Table 1 – Cross-sections

| Story | Columns | | Beams |
|---|---|---|---|
| | 1 and 4 | 2 and 3 | |
| 6th | 30x60 | 30x60 | 30x50 |
| 5th | 30x60 | 30x60 | 30x50 |
| 4th | 30x60 | 30x60 | 30x50 |
| 3rd | 30x60 | 30x60 | 30x60 |
| 2nd | 30x60 | 30x60 | 30x60 |
| 1st | 30x60 | 30x60 | 30x60 |

acceleration spectrum normalized with respect to g), and the total seismic weight of the building $W$:

$$F_h = C\,R\,W \tag{12}$$

Assuming a low seismicity site, the seismic coefficient was set equal to 0.04; 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 is the outermost frame along $y$-direction; its geometrical scheme is shown in Fig. 2b and the cross-section sizes of the members are presented in Table 1. Due to the presence of the flat beams, the four internal frames orientated along $y$-direction provide a negligible contribution to lateral strength and stiffness. Thus, the considered frame is designed to sustain one fourth of the total seismic force. 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 [5]; however the cross-sections of columns are selected not smaller than those of beams, to avoid excessive concentration of damage in one story. In this regard, the characteristic compressive cubic strength $R_{ck}$ is assumed equal to 25 MPa (corresponding to cylinder strength $f_{ck}$ equal to 20 MPa) for concrete and a steel grade Feb44k with a characteristic yield strength $f_{yk}$ = 430 MPa for rebars.

## 3.2 Numerical model adopted for the nonlinear dynamic analysis

A two-dimensional numerical model with masses concentrated at the floor levels is used to evaluate the nonlinear response of the analyzed structures. It is assumed that the actual seismic weight is 25% larger than the original value, because of modifications of type of occupancy. A leaning column is included in the numerical model to account for the $P$-$\Delta$ effects on the lateral system from the gravity columns in the rest of the building. The nominal dead loads plus quasi-permanent live loads are assigned as initial gravity loads in the analysis. The gravity load applied to the leaning column is equal to the weight of the numerical model minus that applied directly to the RC frame. A Rayleigh viscous damping is used and set at 5% for the first and the third mode of vibration. The $P$-$\Delta$ effect is considered in the analysis. All the nodes of the same floor, included that of the leaning column, are constrained to have the same horizontal displacement, in order to simulate the rigid diaphragm effect due to the concrete deck.

A member-by-member modelling with beam with hinges elements is adopted for beams and columns. In particular, the “Beam With Hinges Element” implemented in OpenSees is used, and beams and columns are modelled as members constituted by an elastic element with plastic hinges at their ends. The length of the plastic

Table 2 – Cross-sections characterization of materials for the dynamic analysis of the frames

| **Concrete** | | **Rebars** | |
|---|---|---|---|
| Cylinder Compressive strength | 20 MPa | Yielding strength | 450 MPa |
| Young’s modulus | 27085 MPa | Young’s modulus | 210000 MPa |
| Strain at maximum strength | $2 \times 10^{-3}$ | Ultimate strain in tension | $7.5 \times 10^{-5}$ |
| Tensile strength in tension | 2.21 MPa | Strain-hardening ratio | 0.0058 |

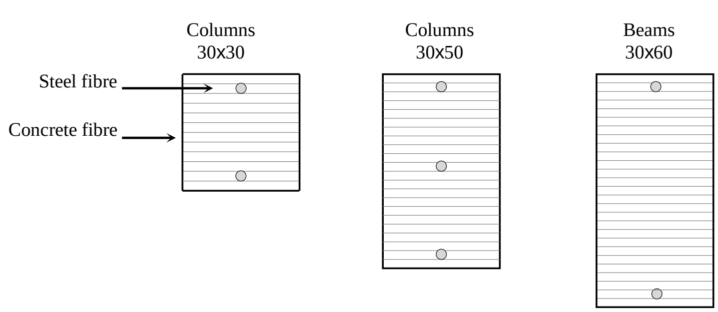



Fig. 3 – Fibre discretization of cross-sections

hinge is equal to the depth of the cross-section. A fiber cross-section is assigned to each plastic hinge, where both concrete and steel components are considered. The concrete part of the cross-section is subdivided in fibers having 5 mm depth and width equal to the width of the section. Single fibers enclosed in the cross-section are used to model rebars. Fig. 3 shows the fiber discretization of cross-sections. The Mander constitutive law is assigned to concrete fibers. An elasto-plastic with strain kinematic hardening constitutive law is assigned to steel fibers. The parameters used for materials are summarized in Table 2. The strain at crushing strength of concrete is assumed very large ($5\times10^{-2}$) in order to avoid numerical instability. The area, the moment of inertia of concrete cross-section and the Young's modulus of concrete are assigned to the elastic element. The "Concrete04" and "Steel01" uniaxial materials implemented in OpenSees are adopted to simulate the cyclic behavior of concrete and steel fibers, respectively.

A "ZeroLength Element" is added at one end of each beam. This element connects the end of the beam to the corresponding node restrained by the rigid deck and is characterized by a large axial deformability. This expedient allows the beams to deform axially and avoids arising of axial force, which typically leads RC beams modelled by fiber elements to an artificial stiffening and strengthening. Furthermore, large shear and flexural stiffnesses are assigned to the ZeroLength Element to transfer shear force and bending moment from the beam to the frame node.

In case of the upgraded frame, the numerical model includes also BRBs, which are modelled as truss elements with the cross-sectional area equal to equivalent area $A_{B,eq}$. The cyclic behavior is simulated by the material model proposed by Zona and Dall'Asta [7] for steel buckling restrained braces, which allows a gradual variation of the axial stiffness of braces and reproduces both kinematic and isotropic hardening [8]. The stiffness properties of this model are defined by the initial elastic stiffness $k_0$, and the post-yield stiffness $k_1$, which are provided by the following equations

$$k_0 = E_s , \qquad k_1 = k_h k_0 \tag{13}$$

where $k_h$ is the kinematic strain hardening ratio assumed equal to 3.16%. The strength of the material is defined by the yield stress $f_{y,eq}$, the maximum yield stress in tension for the fully saturated isotropic hardening condition $f_{y,max}$ and the maximum yield stress in compression for the fully saturated isotropic hardening condition $f_{y,min}$

$$f_{y,eq} = \frac{N_{B,y}}{A_{B,eq}} , \qquad f_{y,\max} = 1.15\ f_{y,eq} , \qquad f_{y,\min} = 1.15\ \beta\ f_{y,eq} \tag{14}$$

being the compression strength adjustment factor β assumed equal to 1.10, based on results of experimental tests [9]. Finally, the coefficient δ, which rules the rate of the isotropic hardening, and the coefficient α, which controls the transition from the elastic to the plastic response, are set as follows

$$\delta = 0.20 , \quad \alpha = 0.6 \tag{15}$$

## 5. Application of the design method to the case study

The proposed design method has been applied to determine the axial stiffness and the yield strength of BRBs for the seismic upgrading of the presented case study. The elastic numerical model adopted for the determination of displacement and strength demands simulates beams and columns by De Saint Venant members, and BRBs by truss elements. The modal response spectrum analysis evaluates the effect of earthquake excitation. The seismic input is given by the elastic spectrum proposed by the EC8 for soil type C and a reference peak ground acceleration set according to the considered limit state. Pushover analysis for the determination of lateral yield strength of the frame is performed by a vertical distributions of lateral loads proportional to the first mode of vibration of the frame. A member-by-member modelling is adopted for pushover analysis. Beams and columns are modelled by elastic members with rigid-plastic hinges assigned at their ends. BRBs are modelled by elastic-plastic truss elements with kinematic hardening. The cross-section area of BRBs is assumed constant along the member and equal to equivalent area $A_{B,eq}$. Based on previous studies by the authors [10], the post-yield stiffness ratio $k_h$, which accounts for kinematic hardening, is set equal to 3.16%. Based on the same studies, the effect of isotropic hardening is considered assuming a fictitious yield axial force equal to 1.15 times the nominal value

$N_{B,y}$. The drift capacity of the RC frame is evaluated considering the provisions of EC8 [2] for the evaluation of the ultimate chord rotation $\theta_{um}$ and assuming the material strengths reduced by the partial factor of the material, equal to 1.5 and 1.15 for concrete and steel, respectively. The ductility capacity of BRBs $\mu_{B,LS}$ is assumed equal to 25. To examine the influence of the design parameters on the required features of BRBs, the design method has been applied considering three values of design story drift $\Delta_d$, i.e. 1.0, 0.8 and 0.6 times the drift capacity $\Delta_{LS}$ of the RC frame. For each value of $\Delta_d$ the behavior factor $q$ ranges from 3 to 9 in steps of 2. In each of these cases, the design procedure assumed the Near Collapse (NC) limit state as target limit. According to the EC8-Part 1 and the relevant National Annex issued in Italy [11], for this limit state the minimum seismic excitation level is associated to the 5% probability of exceedance in 50 years, i.e. a minimum capacity in terms of PGA equal to 0.45 g.

The performance of the frame upgraded with different values of design parameters is evaluated by means of nonlinear dynamic analysis and the results compared to those of the bare frame. The numerical model defined in Section 3.2 is adopted. A set of ten artificial ground motions, compatible with the EC8 elastic spectrum for soil type C and characterized by 5% damping ratio is adopted as seismic input. The SIMQKE computer program [12] 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 [13]. Since the target seismic level assumed in this case was the NC state, this reference suite of ground motions is scaled to the PGA of 0.45 *g*.

The performance of the bare frame is compared to that of the upgraded frame in terms of distribution along the height of the ratio of drift demand to capacity $\Delta/\Delta_{LS}$. The maximum value of the ratio $\Delta/\Delta_{LS}$ along the height of the frame is assumed as representative of its seismic performance and is larger than 1 for frames that exceed the target limit state. For each value of $\Delta_d$ assumed in the design, the seismic response of the bare frame (black circle points) is compared to that of the rehabilitated frames designed considering different values of $q$ (white, grey and black squared points). In particular, Fig. 4a, 4b and 4c show the heightwise distribution of $\Delta/\Delta_{LS}$ for $\Delta_d$ equal to 1.0, 0.8 and 0.6 of $\Delta_{LS}$, respectively. Focusing on the bare frame, at the considered limit state, it suffers from a severe damage concentration at the second story, and the drift demand strongly overcomes its capacity at almost all the levels (the ratio $\Delta/\Delta_{LS}$ is larger than 1). The results point out that insertion of BRBs modifies the distribution along the height of the ratio $\Delta/\Delta_{LS}$, reducing the damage concentrations. In particular, the reduction of the story drift mainly depends on the assumed value of drift design $\Delta_d$. If the design story drift $\Delta_d$ is assumed equal to the drift capacity $\Delta_{LS}$, the response of the upgraded frame slightly improves, but the drift demand is not yet reduced below an acceptable value, as displayed in Fig. 4a. If the upgraded frame is designed assuming $\Delta_d$=0.8 $\Delta_{LS}$ (Fig. 4b) the retrofitting intervention becomes more effective and BRBs reduces the story drift capacity of the frame. However, the ratio $\Delta/\Delta_{LS}$ is still generally larger than 1 and the frame does not meet the target seismic performance yet. When $\Delta_d$ is set equal to 0.6 $\Delta_{LS}$ for the BRBs design, all the frames upgraded with different values of $q$ satisfy the requirements of the NC limit state verification (Fig. 4c). Moreover, the value assumed for the behavior factor $q$ does not show a strong influence on the final response of the upgraded frame.

These results shows that when the requirement on the design story drift is more restrictive, i.e. $\Delta_d$ is taken

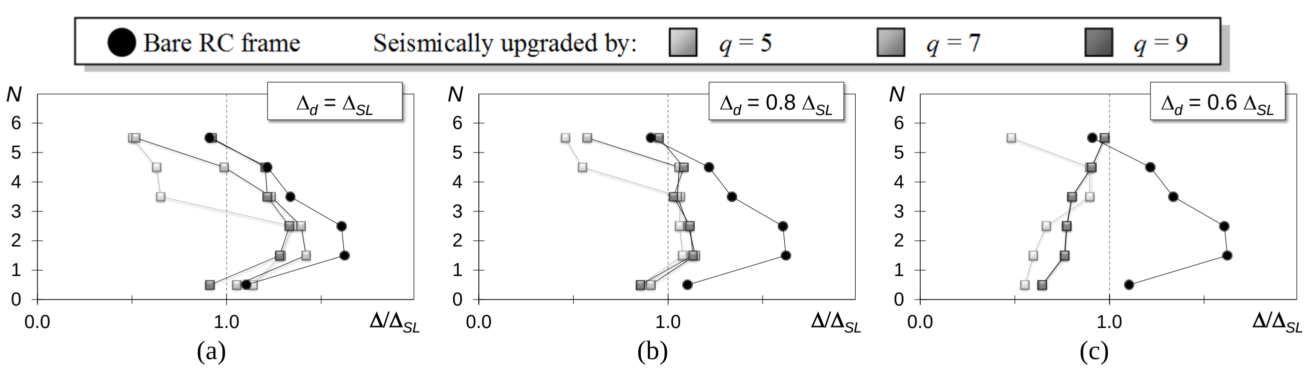



Fig. 4 – Effects of seismic upgrading on drift demand to capacity ratio

as a smaller percentage of the drift capacity $\Delta_{LS}$, the design leads to stiffer and stronger BRBs. This means that choosing the appropriate values of the design parameters, the design method is able to determine the proper stiffness and strength of BRBs, to reduce the drift demand below the capacity and promote a more favorable collapse mechanism. Particularly, in this case the value suggested for the design story drift is 0.6 times the drift capacity corresponding the considered limit state.

## 6. Conclusions

This paper presents a design method of BRBs for the seismic upgrading of existing RC frames. The proposed design procedure determines the stiffness and strength of BRBs in order to (i) reduce the drift demand of the bare frame below its capacity and (ii) promote a uniform distribution along the height of the frame of the ratio of lateral strength over seismic shear demand. To this end, the method develops into two main steps and three requirements have to be fulfilled: the drift demand must be lower than the design drift $\Delta_d$; the ductility demand of BRBs must be lower than their ductility capacity; the total lateral strength of the upgraded frame has to be equal to a certain target level. The behavior factor $q$ and the design story drift $\Delta_d$ are the ruling parameters.

The design procedure has been applied to a six-story three-bay RC frame, representative of the RC structures designed in Italy according to old seismic codes and for low seismicity area. The BRBs were designed so that the upgraded frame could sustain ground motions having the probability of exceedance of 5% in 50 years, i.e. assuming as target limit state the NC state. The design of the BRBs have been repeated, considering three values of design story drift $\Delta_d$, i.e. 1.0, 0.8 and 0.6 times the drift capacity $\Delta_{LS}$ of the RC frame. For each value of $\Delta_d$ the behavior factor $q$ ranged from 5 to 9 in steps of 2. The performance of the bare frame at the considered limit state was compared to that of the upgraded frames by means of nonlinear dynamic analysis. The seismic response was considered in terms of distribution along the height of the ratio of drift demand over drift capacity.

The investigation presented in this paper shows the efficiency of the proposed design method. Particularly, it was found that the drift requirement influences the effectiveness of the retrofitting intervention, more than the strength requirement. In fact, the numerical results showed that the design method applied with $\Delta_d = 0.6\ \Delta_{LS}$ led to the most effective results, almost independently on the values of the behavior factor. In this case, the drift demand was lower than the drift capacity at each story for all the considered behavior factors. Furthermore, in all cases the mechanism became more distributed, and the damage concentration experienced by the bare frame was mitigated.

## 7. References

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