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CALIBRATION OF A DESIGN METHOD FOR SEISMIC UPGRADING OF EXISTING R.C. FRAMES BY BRBS

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ABSTRACT

In this paper, the introduction of Buckling Restrained Braces (BRBs) is proposed for the seismic rehabilitation of existing r.c. framed structures. Furthermore, a procedure for the design of BRBs is presented. According to this procedure, BRBs are designed to fulfil both a stiffness and a strength requirement, in order to achieve the Significant Damage limit state in occurrence of seismic events having a 10% probability of exceedance in 50 years. The parameters that rule the design are the behaviour factor q and the design storey drift Δ_d

The effectiveness of the proposed method has been verified evaluating the seismic response of an r.c. frame upgraded by BRBs, which were designed using different pairs of values of Δ_d and q. The seismic response of the obtained frames has been assessed by nonlinear dynamic analysis, whose results demonstrate that the seismic performance of the upgraded frames is suitable as long as proper values of Δ_d and q are adopted.

INTRODUCTION

Although many regions of the world are subjected to frequent seismic activities, a large part of existing r.c. buildings had been designed before seismic codes entered into force. These structures do not accomplish seismic design criteria, because they were designed to sustain gravity loads only. They usually have main structural elements, columns and beams, disposed along a single direction and this makes these structures very flexible and weak in the orthogonal direction. Furthermore, the distributions of the lateral stiffness and shear strength along the height of these frames are not suitable to make the displacement/deformation demand widespread along the height and consistent with the capacity of the frame.

In this paper, the introduction of Buckling Restrained Braces (BRBs) is proposed for the seismic rehabilitation of existing r.c. framed structures designed to sustain gravity loads only. BRBs typically consist of a ductile steel core confined by a steel tube (Uang and Nakashima, 2004; Xie, 2005). In some cases the tube is filled with mortar and an unbonding layer is interposed between the steel core and the mortar so that the central yielding core can deform longitudinally without interaction with the jacket that restrains lateral and local buckling. The brace is joined to the frame by means of the connection segments, while the yielding core and the connection segments are linked by the

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transition segments. The transition and connection segments have to remain elastic during cyclic loading. This is obtained by adopting for these segments cross-sectional areas A_t and A_j larger than that of the yielding core A_c. The insertion of BRBs can increase to proper value both the lateral stiffness and the shear strength of the structure. Furthermore, it 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 favourable collapse mechanism during strong ground motions. Finally, it can modify the distribution of the lateral stiffness along the height so that the displacements demanded by the ground motion can better fit the displacement capacity of the structure.

This research aims at firstly developing, and then validating, a design method for BRBs, whose basic idea was already presented by the authors for the design of steel frames with BRBs (Bosco and Marino, 2013; Bosco *et al.*, 2014). The design procedure permits to define at each storey the 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 parameters that rule the design are the behaviour factor q and the design storey drift Δ_d, which control the lateral strength and stiffness provided to structure, respectively. According to this method, the r.c. frames upgraded by BRBs have to fulfil both a displacement and a strength requirement. The fulfilment of the first requirement reduces the storey drift demand to values compatible with the structure deformation capacity, while the fulfilment of the second one increases the shear strength of the structure and distributes it along the height more favourable for seismic purposes.

In the first part of the paper, the proposed design procedure is presented and applied for the seismic upgrading of an r.c. frame designed considering gravity loads only. BRBs are designed by means of two values of the design storey drift and three values of the behaviour factor q. In the second part of the paper, the frames upgraded by BRBs are subjected to a set of ten artificial accelerograms. The seismic response is determined by nonlinear dynamic analysis and represented in terms of the distribution along the height of the drift demand to capacity ratio of the r.c. frame. Based on this preliminary parametrical analysis, the largest behaviour factor corresponding to drift demand to capacity ratio not larger than one is proposed for the design of BRBs.

PROPOSED DESIGN METHOD

The proposed design method aims at attaining a dissipative collapse mechanism and avoiding the exceeding of the design limit state (corresponding to a given level of damage or, eventually, to the collapse of the structure) in occurrence of the relevant ground motion. The method deals with two specific requirements. The first one is a displacement requirement aiming at reducing the displacement demand below the design value. The second one is a strength requirement and aims at providing the r.c. frame upgraded by BRBs with sufficient lateral strength, which is distributed along the height of the frame proportionally to the storey shear demanded by the design ground motion. This promotes the yielding of the BRBs at all storeys and, therefore, the achievement of a dissipative collapse mechanism.

The method is ruled by two parameters. The first parameter is the *design storey drift* $\Delta_{d,i}$, which is given as a fraction of storey drift capacity $\Delta_{l,i}$. The storey drift $\Delta_{d,i}$ is assumed smaller than the nominal capacity $\Delta_{l,i}$ to take into account that the storey drifts demanded by the earthquake may be larger than those obtained by the design elastic analysis due to some concentrations of the plastic deformations in a few storeys. The second parameter is the *behaviour factor q*, which determines the lateral strength to be provided by BRBs. Given the value of the ruling parameters, the procedure starts with the fulfilment of the displacement requirement, which requires reducing the storey drifts demanded by the earthquake below the design storey drifts. To this end, the method determines the additional stiffness to be provided by BRBs. After that, the procedure focuses on the strength requirement and determines the additional strength to be provided by BRBs to make the lateral resistance of the structure equal to that required by the design analysis.

Determination of the displacement and strength demands

A reliable evaluation of the displacement demand of the frame due to strong ground motions should be obtained by a nonlinear method of analysis able to predict the possible concentration of the inelastic deformation. However, because of the insertion of BRBs promotes the simultaneous yielding of all the storeys, the displacement demand is expected to be widespread along the height. Hence, the demanded storey drifts Δ_i are determined by the elastic analysis of the structure, based on the elastic (unreduced) spectrum of the reference ground motion, here assumed as that having a 10% probability of exceedance in 50 years. Finally, Δ_i is modified by multiplying it for the coefficient q_d, which takes into account that the equal displacement rule does not apply for structures whose fundamental period T_1 is smaller than T_C. According to Eurocode 8, the coefficient q_d is determined as:

$$q_d = \frac{1}{q}\left[1+(q-1)\frac{T_C}{T_1}\right] \tag{1}$$

The required lateral strength of the frame is determined at each storey as the storey shear $V_{Ed,i}$ evaluated by the elastic analysis of the structure based on the elastic spectrum of the reference ground motion reduced by means of the behaviour factor q. This second elastic analysis is performed after that the BRBs are inserted within the frame and their stiffness has been determined in compliance to the displacement requirement.

Determination of the displacement capacity

In this paper, the displacement capacity is defined in terms of storey drifts corresponding to the achievement of the Significant Damage limit state in columns. The provisions of the European seismic code (EC8, 2004) are adopted to define this limit state. In particular, this code quantifies the seismic performance in terms of chord rotation, i.e. the angle between the tangent to the axis at the yielding end and the chord connecting this end with the point of contraflexure. The limit value of chord rotation θ_{um} corresponding to Near Collapse limit state can be evaluated by the following equation:

$$\theta_{um} = \frac{1}{\gamma_{el}} 0.016\left(0.3^{\nu}\right)\left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)} f_c\right]^{0.225}\left(\frac{L_V}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)} \tag{2}$$

where γ_{el} is equal to 1.5, f_c and f_{yw} are the mean values of the concrete compressive strength and of the yield stress of the stirrup divided by the confidence factor *FC* and partial safety factors, *h* is the depth of cross-section, ω and ω' are the mechanical ratio of the tension and compression longitudinal reinforcement, respectively. The parameter ν is calculated as the axial force *N* normalised with respect to the resistance in compression of the concrete section ($A_c f_c$), and $L_V = M/V$ is the ratio moment/shear at the end section. Finally, Equation (2) takes into account the effect of the confinement due to reinforcements (longitudinal bars and stirrups) by the confinement effectiveness factor α and steel ratio of transverse reinforcement ρ_{sx}.

Furthermore, EC8 defines the chord rotation capacity θ_{um} as the chord rotation at yielding θ_y plus plastic rotation at column failure θ_{um}^{pl} and stipulates that the Significant Damage limit state is achieved when the plastic part of the chord rotation is equal to 75% of θ_{um}^{pl} is attained somewhere in the structure. The plastic rotation at column failure θ_{um}^{pl} is calculated as:

$$\theta_{um}^{pl} = \frac{1}{\gamma_{el}} 0.0145\left(0.25^{\nu}\right)\left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)}\right]^{0.3} f_c^{\,0.2}\left(\frac{L_V}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)} \tag{3}$$

and the chord rotation at yielding θ_y is obtained as difference between the value of the total chord rotation capacity θ_{um} and plastic rotation at column failure θ_{um}^{pl}. Finally, the displacement capacity Δ_l corresponding to the Significant Damage limit state is evaluated as:

$$\Delta_l = \left[0.75\cdot\theta_{um}^{pl} + (\theta_{um} - \theta_{um}^{pl})\right]\cdot H \tag{4}$$

where H is the length of the column equal to the inter-storey height of the frame. The drift Δ_l is evaluated for the two ends of all the columns of the storey and the minimum value obtained is assumed as displacement capacity of the storey $\varphi_{l,i}$.

Determination of the available strength

The available strength $V_{Rd,i}$ at each storey of the frame upgraded by BRBs is evaluated as the summation of two contributions: the shear strength of the bare r.c. frame $V_{Rd,f,i}$ plus the storey shear strength provided by BRBs $V_{Rd,b,i}$:

$$V_{Rd,i} = V_{Rd,f,i} + V_{Rd,b,i} \tag{5}$$

The storey shear strength of the bare r.c. frame $V_{Rd,f,i}$ is given by the sum of the shear forces transmitted by the columns of that storey when they are yielded in flexure at the two end cross-sections. Given this definition of $V_{Rd,f,i}$, the storey shear strength of the bare r.c. frame may be evaluated by the equations proposed by Bosco *et al*. (2008). In this paper, as an alternative to these equations, $V_{Rd,f,i}$ is calculated by a pushover analysis of the frame in which the actual strength is assigned to the columns of the i-th storey and infinite strength is assigned to all the other members. In this way, the yielding of the columns of the i-th storey is obtained and the sum of the shear forces of the columns at the i-th storey provides $V_{Rd,f,i}$. The storey shear strength of the BRBs $V_{Rd,b,i}$ is calculated as the sum of the two horizontal components of the axial forces of the braces inserted in the frame at the considered storey:

$$V_{Rd,b,i} = 2\ N_{y,i}\ \cos\alpha \tag{6}$$

where $N_{y,i}$ is the yield strength of the BRB at the i-th storey and α is the angle of inclination of the BRBs with respect to the beam longitudinal axis.

Design of BRBs

BRBs are designed firstly to fulfil the displacement requirement. To this end, the required drift and the drift capacity are compared each other at every storey. Where the drift demand exceeds the capacity, the introduction of BRBs is required to provide the structure with the lacking stiffness.

With regards to the total stiffness of the structure K_{req}, it is given by the stiffness of the bare r.c. frame model $K_{FR,b}$ (Fig. 1a) and the stiffness of the truss model K_{Truss} (Fig. 1b):

$$K_{req} = K_{FR,b} + K_{Truss} \tag{7}$$

The total required stiffness K_{req} is evaluated as the ratio between the total shear force and design storey drift Δ_d divided by the coefficient q_d. The bare r.c. frame stiffness $K_{FR,b}$ is calculated as the ratio of the sum of the shear forces carried by the columns over the storey drift. Both the storey shear force and drift are determined by the elastic analysis with the unreduced spectrum for the determination displacement demand. From Equation (7), the stiffness K_{Truss} can be calculated as K_{req} minus $K_{Fr,b}$.

Furthermore, it is considered that the total storey drift of the truss model Δ is equal to the sum of the drift caused by the axial deformations of BRBs Δ_{BRBs} (Fig. 2a) and that caused by the axial deformation of columns Δ_{COLax} (Fig. 2b). The total storey drift of the truss model Δ is calculated as the ratio between the shear force sustained by BRBs V_{BRBs}, determined by the elastic analysis, and the stiffness K_{Truss}. Focusing on the drift caused by the axial deformation of columns (Fig 2b), it is evident that shortening/elongation of columns would cause also a variation in length of BRBs. But, to avoid this, columns rotate rigidly (Fig. 2b) and provide the drift Δ_{COLax} that is determined by simple geometrical considerations. Finally, the drift Δ_{BRBs} is calculated as Δ minus Δ_{COLax} and the additional stiffness to be provided by BRBs to fulfil the displacement requirement is determined:

$$K_{BRBs} = \frac{V_{BRBs}}{\Delta_{BRBs}} \tag{8}$$

Given the value of K_{BRBs}, the procedure determines the value of the *equivalen*t cross-section area of BRBs A_{eq} by the following relation:

$$A_{eq} = \frac{1}{2}\frac{K_{BRBs}\ L_{BRBs}}{E\cos^2\alpha} \tag{9}$$

where L_{BRBs} is the length of BRBs and E is Youngôs modulus of steel. It is noteworthy to point out that the equivalent area A_{eq} can be obtained adjusting the length and the cross-section of the parts of the BRBs and, therefore, by several configurations of the BRB (Xie, 2005; Bosco and Marino, 2013). Because the insertion of the BRBs increases the frame stiffness, and modifies the periods and the seismic response of the frame, the procedure for the design of the equivalent cross-section area of BRBs has to be ran iteratively until convergence.

After that the stiffness of BRBs is calculated, their yield strength N_y is determined to fulfil the strength requirement. To this end, the required strength of the BRBs at the *i*-th storey, $V_{reqBRBs,i}$, is obtained as the difference between the total required strength $V_{Ed,i}$ (obtained by the elastic analysis of the frame with the spectrum reduced by q) and the shear strength of the bare r.c. frame $V_{Rd,f,i}$:

$$V_{req,BRBs,i} = V_{Ed,i} - V_{Rd,f,i} \tag{10}$$

and the yield strength of the BRBs is calculated by the following relation:

$$N_{y,i} = \frac{V_{req,BRBs,i}}{2\cos\alpha} \tag{11}$$

For each of the BRB configuration that satisfies the displacement requirement considered in the first phase of the proposed deign procedure, the yield strength N_y can be obtained adopting a proper yield stress:

$$f_y = \frac{N_y}{A_c} \tag{12}$$

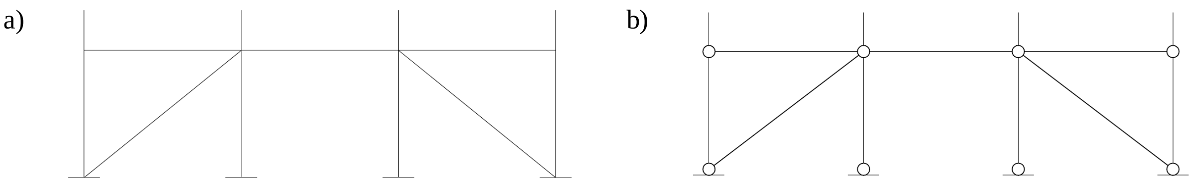


Figure 1. Models for evaluation of the contributions to the frame stiffness: a) bare frame model, b) truss model

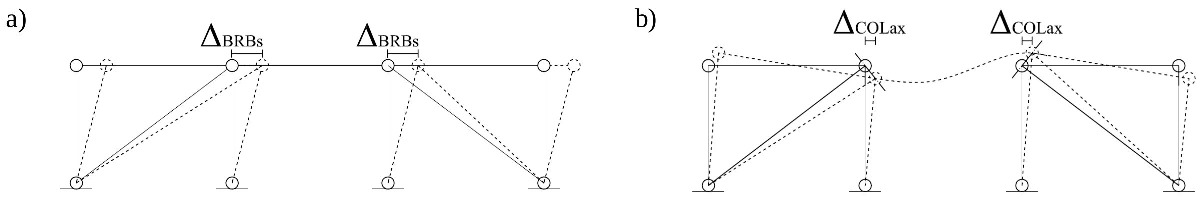


Figure 2. Drifts caused by a) BRBs axial deformation and b) columns axial deformation

where A_c is the cross-sectional area of the yielding core of the BRB. If one of the obtained values of f_y falls within an acceptable range of yield stresses, defined by a minimun and maximum values $f_{y,min}$ and $f_{y,max}$, it can be adopted and the design of the BRB will result optimal. If f_y is always smaller than $f_{y,min}$, the value $f_{y,min}$ has to be adopted and the BRB will be oversized for strength. Finally, if f_y is always larger than $f_{y,max}$ the value $f_{y,max}$ has to be adopted and the cross-section of the BRB has to be enlarged. In this case the BRB will be oversized for stiffness.

ANALYSED STRUCTURE

The previously described design method has been applied for the seismic upgrading of a r.c. frame. This latter one was drawn from a building representative of many r.c. framed structures designed to resist gravity loads only according to the Italian regulations in force during the Seventies (Italian Ministry of Public Works, 1971 and 1974). The building is characterized along *x*-direction by four frames made of seven spans, while in *y*-direction it has two frames of three spans and two frames of one span (Fig. 3). Because the majority of the structural elements are arranged along the *x*-direction, the structure results weak and flexible for earthquakes directed along *y*-direction. For this reason, the proposed design method has been applied to the outermost frames oriented along the *y*-direction.

The analysed frame is six-storey high and its geometrical scheme is shown in Figure 4, together with the arrangement of BRBs within the spans. Moreover, the table included in the figure describes the size of the structural members. The BRBs are designed supposing that this frame sustains 30% of the total seismic force. Considering all the gravity loads applied on the structure, the floor mass has been considered equal to 102.37 t. The gravity loads have been calculated as seismic combination of dead loads g_k and live loads $\psi_2\, q_k$ for the seismic design situation. The elastic numerical model

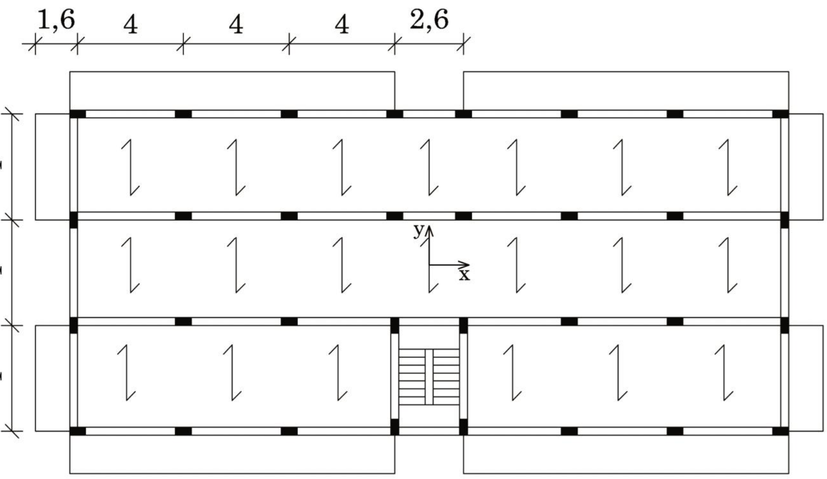
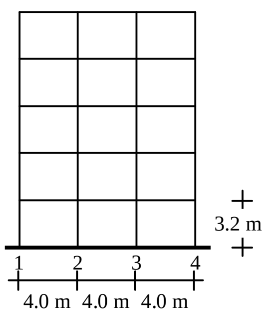


Figure 3. Plan of the building design for gravity loads

(a)



(b)

Cross-section dimensions (cm)

Story	Col. 1 and 4	Col. 2 and 3	Beams
6th	30 x 30	30 x 30	30 x 60
5th	30 x 30	30 x 30	30 x 60
4th	30 x 30	30 x 30	30 x 60
3rd	35 x 30	30 x 40	30 x 60
2nd	40 x 30	30 x 50	30 x 60
1st	50 x 30	30 x 60	30 x 60

(c)

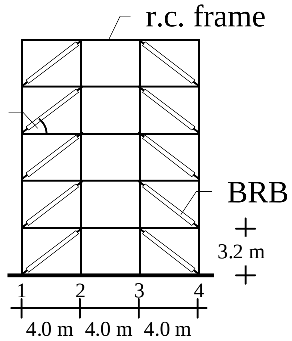


Figure 4. Analysed frame: (a) geometrical scheme of the bare r.c. frame, (b) cross-sections dimensions (b x h) of the r.c. frame members, (c) arrangement of the BRBs within the r.c. frame

adopted for the determination of displacement and strength demands simulates beams and columns by De Saint Venant members. All the nodes belonging to the same floor are constrained to have the same horizontal displacement. BRBs are modelled as trusses. The modal response spectrum analysis with SRSS combination method is used to evaluate of the earthquake excitation. The seismic input is given by the elastic spectrum proposed by the EC8 for soil type C, characterized by a peak ground acceleration a_g equal to 0.35 g. This earthquake level is suggested by the Italian national annexes to EC8 for the design of structures located in high seismicity areas.

Beams and columns are realized with concrete having a characteristic compressive cylinder strength f_{ck} equal to 21 MPa, Youngôs modulus E_{cm} = 30280 MPa, and Poissonôs ratio equal to 0.5. The transverse and longitudinal reinforcements consist of deformed bars (steel type FeB38k) with a characteristic yield stress f_{yk} equal to 375 MPa, Youngôs modulus E_s= 210000 MPa, and Poissonôs ratio equal to 0.3. The mean values of the compressive strength of concrete and yield stress of steel reinforcement are assumed to equal f_{cm} = 29 MPa and f_{ym} = 400 MPa, respectively. The confidence factor FC is assumed equal to one for both concrete and steel. The Youngôs modulus and Poissonôs ratio of the steel adopted for BRBs are assumed equal to those of the steel reinforcement. Instead the yield stress of BRBs f_y is determined by the proposed design procedure within a range of values deemed acceptable bounded by $f_{y,min}$ = 100 MPa and $f_{y,max}$ = 275 MPa.

The displacement capacity $\Delta_{l,i}$ is evaluated for each storey considering the axial force of columns equal to that provided by gravity loads. This axial force is evaluated based on the concept of tributary areas and load per square meter corresponding to the seismic design situation. The moment/shear ratio L_V is calculated considering that both the end cross-sections of column are yielded. The concrete compressive strength and the stirrup yield stress are assumed equal to their mean values. The lateral strength of the bare r.c. frame $V_{Rd,f,i}$ is calculated at each storey by pushover analysis. A load pattern with horizontal forces proportional to the first mode of vibration is adopted. Columns of the i-th storey are modelled by beam-column elements with concentrated plasticity assigned at member ends. An elasto-plastic behaviour is assumed for the plastic hinges with plastic bending moment equal to the flexural strength of the column in occurrence of the axial force provided by gravity loads. This flexural strength is determined based on the mean values of the compressive strength of concrete and yield stress of longitudinal reinforcement. BRBs of the storey are modelled as trusses with elasto-plastic behaviour and kinematic strain hardening. The kinematic strain hardening is calibrated so that the ratio N/N_y is equal to 1.15 and 1.59 when the BRB ductility demand is equal to 1 and 15, respectively. This kinematic strain hardening also includes the effects of isotropic hardening (Bosco *et al*., 2014). An elastic behaviour is assumed for all the other members. The pushover analysis is repeated 6 times changing the storey where the plastic mechanism takes place. Both the elastic and pushover analyses have been carried out by means of the program TEL2008 (Ghersi, 2014).

The design of the analysed frame is carried out three times considering two design storey drifts $\Delta_{d,i}$ equal to 0.7 $\Delta_{l,i}$ and 0.8 $\Delta_{l,i}$ and three values of the behaviour factor q equal to 5, 6 and 7.

NUMERICAL ANALYSES

The seismic response of the bare r.c. frame and that of the r.c. frames upgraded by BRBs is evaluated by nonlinear dynamic analysis. Numerical analyses are carried out by means of the OpenSees program (Mazzoni *et al*. 2003). The results obtained are used to investigate the influence of the parameters governing the design procedure (the behaviour factor q and the design story drift Δ_d) on the seismic performance of the frame and to determine the values of q and Δ_d leading to frames that do not exceed the Significant Damage Limit State for earthquake excitation corresponding to 10% probability of exceedance in 50 years.

Numerical model

A two-dimensional frame model with masses concentrated at the floor levels is used to evaluate the nonlinear response of the analysed structures. Columns of the first storey are fully restrained at their base. In compliance with EC8, the nominal dead loads plus quasi-permanent live loads are assigned as initial gravity loads in the analysis. A Rayleigh viscous damping is used and set at 5% for the first and the third mode of vibration. The P-φ effect is considered in the analysis. All the nodes of the same floor 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 ñ*Beam With Hinges Element*ò implemented in OpenSees is used, and beams and columns of the r.c. frame are modelled as members constituted by an elastic element with plastic hinges at their ends. The length of the plastic hinge is equal to the depth of the cross-section. A fibre 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 fibres having 5 mm depth and width equal to the width of the section. Single fibres enclosed in the cross-section are used to model the rebars. Figure 5 shows the fibre discretization for some cross-sections. The Mander constitutive law is assigned to the concrete fibres. The confinement effect of stirrups is neglected and the compressive strength and the Youngôs modulus of concrete are assumed equal to their mean values f_{cm} = 29 MPa and E_{cm} = 30280 MPa. The strain at maximum strength is equal to $2\text{x}10^{-3}$. The strain at crushing strength is assumed very large ($5\text{x}10^{-2}$) in order to avoid numerical instability. The tensile strength is assumed equal to f_{tm} = 2.28 MPa and the ultimate strain is $7.5\text{x}10^{-5}$. An elasto-plastic with strain kinematic hardening constitutive law is assigned to these fibres. The yielding strength is f_{ym} = 400 MPa, the Youngôs modulus is E_s = 210000 MPa and the strain-hardening ratio is assumed equal to 0.0066. The area, the moment of inertia of concrete cross section and the Youngôs modulus of concrete are assigned to the elastic element. The ñ*Concrete04*ò and ñ*Steel01*ò uniaxial materials implemented in OpenSees are adopted to simulate the cyclic behaviour of concrete and steel fibres, respectively.

A ñ*ZeroLength Element*ò 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 characterised by a large axial deformability. This expedient allows the beams to deform axially and avoids arising of axial force, which typically leads r.c. beams modelled by fibre 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.

BRBs are modelled as truss elements with the cross-sectional area equal to equivalent area A_{eq} obtained in design. The cyclic behaviour is simulated by the material model proposed by Zona and DallôAsta (2012) for steel buckling restrained braces, which allows a gradual variation of the axial stiffness of the brace and considers both kinematic and isotropic hardening. 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

$$k_0 = E_s\,, \qquad\qquad k_1 = k_{\mathrm{h}} k_0 \tag{13}$$

where k_1 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 $f_{y,min}$

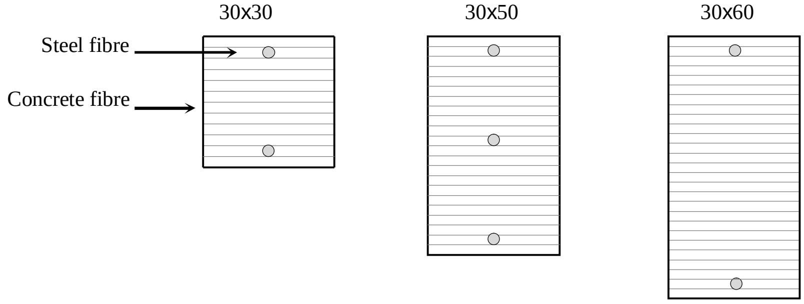


Figure 5. Fibre discretization of cross-sections

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

where б represents the ratio of the maximum force in compression over the maximum force in tension and is assumed equal to 1.10, based on results of experimental tests (Newell *et al.*, 2006). Finally, the coefficient ŭ, which rules the rate of the isotropic hardening, and the coefficient Ŭ, which controls the trend of the transition from the elastic to the plastic response, are set as follows

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

Accelerograms

A reference set of ten artificial accelerograms, compatible with the EC8 elastic spectrum for soil type C and characterized by 5% damping ratio and peak ground acceleration a_g equal to 0.35 g, is defined for nonlinear time-history analysis. Each accelerogram of the reference set is characterised by a total duration of 20 s and is enveloped by a ñcompoundò function that presents three branches: the first branch is an exponential increasing function, the second is a constant function (*strong motion phase*), and the third is a function with exponential decay. The duration of the strong motion phase of the accelerogram is equal to 7.0 s and thus lower than the minimum value suggested by EC8, i.e. 10 s. The adopted value has resulted from a previous investigation in which natural and artificial accelerograms were compared in terms of the input energy spectra, Arias intensity, frequency content and number of equivalent cycles. Details about the choice of this envelope intensity function and the procedure for the determination of the lengths of the parts of the compound function may be found in (Amara *et al.*, 2013). The mean of the peak ground accelerations of the generated accelerograms is not lower than the value stipulated by EC8 and no value of the mean response spectrum is lower than 90% of the corresponding value proposed by EC8. The SIMQKE computer program (Gasparini and Vanmarcke, 1976) is used to generate the accelerograms.

Seismic response of the designed frames

The results of nonlinear dynamic analysis are employed to evaluate the seismic performance of the analysed frames expressed in terms of drift demand Δ and of ratio of storey drift demand Δ over capacity Δ_l. In particular, for each accelerogram, the maximum drift demand and drift demand to capacity ratio is determined at each storey. Note that, the internal forces of columns change during the

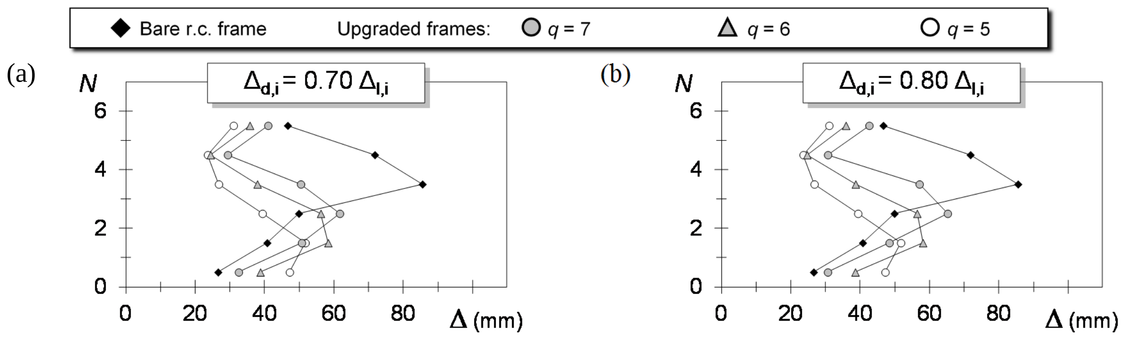


Figure 6. Maximum storey drift demand of the analysed frames: (a) $\Delta_{d,i}$ = 0.7 $\Delta_{l,i}$, (b) $\Delta_{d,i}$ = 0.8 $\Delta_{l,i}$

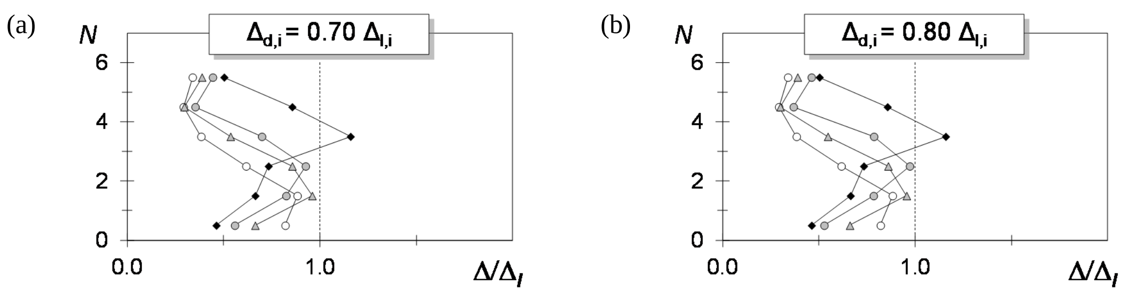


Figure 7. Maximum storey drift demand to capacity ratio of the analysed frames: (a) $\Delta_{d,i}$ = 0.7 $\Delta_{l,i}$, (b) $\Delta_{d,i}$ = 0.8 $\Delta_{l,i}$

earthquake, modifying their ductility capacity and, therefore, the storey drift capacity Δ_l. In order to consider the variation of the storey drift capacity during the earthquake, Δ_l is recalculated at each step of the analysis by Equation (4) considering the current internal forces. For each storey, the mean values Δ_i and $\Delta_i/\Delta_{l,i}$ of the maximum drift and drift demand to capacity ratios are evaluated over the ten accelerograms.

Figure 6 shows the heightwise distribution of the maximum drift demand Δ_i obtained for the bare r.c. frame and for the frames upgraded by BRBs. For each value of $\Delta_{d,i}$ assumed in design, the seismic performance of the bare frame is compared to that of the upgraded frames designed by the considered values of q. In particular, Figures 6a and 6b report the results obtained for $\Delta_{d,i}$ equal to 0.7 $\Delta_{l,i}$ and 0.8 $\Delta_{l,i}$, respectively. The results point out that the insertion of BRBs within the r.c. frame leads to maximum storey drifts along the height smaller than that obtained for the bare r.c. frame. The reduction of the storey drift depends on the behaviour factor adopted for the design of BRBs and is more important when smaller values of q are used. The reduction of the storey drift is also influenced by the design storey drift adopted for BRBs designed by $q = 7$. In this case, the frame designed by $\Delta_{d,i} = 0.7\ \Delta_{l,i}$ is slightly stiffer than that designed by $\Delta_{d,i} = 0.8\ \Delta_{l,i}$ and experiences slightly smaller storey drifts. Instead, when q is assumed equal to 6 or 5, the design of BRBs is controlled only by the strength requirement and the BRBs obtained for $\Delta_{d,i} = 0.7\ \Delta_{l,i}$ and $\Delta_{d,i} = 0.8\ \Delta_{l,i}$ are basically the same. As a consequence, for $q = 6$ or 5, the frames upgraded by BRBs designed with $\Delta_{d,i} = 0.7\ \Delta_{l,i}$ and $\Delta_{d,i} = 0.8\ \Delta_{l,i}$ show the same storey drifts (compare Fig. 6a and Fig. 6b).

Figure 7 shows the seismic response of the analysed frames in the same format of Figure 6, but in terms of the ratio $\Delta_i/\Delta_{l,i}$. The maximum value of the ratio $\Delta_i/\Delta_{l,i}$ obtained along the height of the frame is assumed as representative of its seismic performance and is larger than 1 for frames that exceed the Significant Damage limit state. For an assigned value of Δ_d, the obtained results allow the determination of the proper value to be used for q to achieve the Significant Damage limit state. Focusing on Figure 7, it is notable that the bare r.c frame sustains significant damage concentration at the 4th storey and exceeds the Significant Damage limit state. The insertion of BRBs modifies the distribution along the height of the frame of the ratio $\Delta_i/\Delta_{l,i}$ reducing the damage concentration. Figure 7 shows also that the insertion of BRBs leads always to smaller value of the ratio $\Delta_i/\Delta_{l,i}$ and, therefore, improves the seismic performance. All the pairs of values assumed for $\Delta_{d,i}$ and q lead to frames that do not exceed the Significant Damage limit state (maximum ratio $\Delta_i/\Delta_{l,i}$ smaller than 1). Among the cases considered, the BRBs designed by $\Delta_{d,i} = 0.8\ \Delta_{l,i}$ and $q = 7$ (Fig. 7b) represent the best solution for the seismic upgrading of the analysed r.c. frame from an economical point of view.

CONCLUSIONS

In this paper, a design procedure for the seismic upgrading of existing r.c. frames by BRBs is presented. According to this procedure, ~~the~~ BRBs are designed to fulfil both a stiffness and a strength requirement, in order to promote the achievement of a dissipative collapse mechanism. The method requires the determination of the displacement and strength demands and the evaluation of the displacement capacity and available strength. Then, the BRBs are designed by an iterative procedure to compensate for the lack of stiffness and strength of the frame. Therefore, all the features of the BRBs (cross-section area, length of the yielding segment, yield stress of the steel, etc.) may be determined at each storey of the frame.

The investigation has been conducted on a 6 storey r.c. frame upgraded by BRBs, which were designed by the proposed method using different values for the design parameters, i.e. the design storey drift Δ_d and the behaviour factor q. The results demonstrate that the proposed design method allows the achievement of suitable seismic performance. In particular, if BRBs are designed by proper values of q and Δ_d, the upgraded frame can sustain ground motions having a 10% probability of exceedance in 50 years without exceeding the Significant Damage limit state. In the case study examined, the numerical investigation has also provided the values of q and Δ_d that can be used. In particular, for both the design storey drifts considered $\Delta_{d,i} = 0.8\ \Delta_{l,i}$ and $\Delta_{d,i} = 0.7\ \Delta_{l,i}$, the target limit state is not exceeded even by a behaviour factor equal to 7.

The preliminary investigation presented in this paper evidences that the upgrading of existing r.c. frames by BRBs is a promising technique. In fact, the target seismic performance of the analysed r.c. frame has been achieved by the insertion of BRBs with moderate strength and stiffness. However, it should be pointed out that this conclusion is just based on one case study and, therefore, further investigation is needed to generalise and validate the results of this study.

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