16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 1915 (Abstract ID)

Registration Code: S-C1462389026

SEISMIC UPGRADING OF TYPICAL ITALIAN EXISTING RC FRAMES BY BRBS

F. Barbagallo[(1)], M. Bosco[(2)], E.M. Marino[(3)], P.P. Rossi[(4)], P.R. Stramondo[(5)],

[(1)] *Ph.D. student, University of Catania, fbarbaga@dica.unict.it*

[(2)] *Research Assistant University of Catania, mbosco@dica.unict.it*

[(3)] *Assistant Professor, University of Catania, emarino@dica.unict.it*

[(4)] *Assistant professor, University of Catania, prossi@dica.unict.it*

[(5)] *Doctor researcher, University of Catania, pstramon@dica.unict.it*

Abstract

Although many regions of the world are recognized as earthquake prone areas, a large part of the buildings spread on those territories suffer from structural deficiencies and are not able to sustain horizontal forces. For instance, many structures, which are in use today in Italy, are not adequate to face strong ground motions due to different reasons. A large part of the buildings was designed before seismic provisions became mandatory, taking into account the only effect of gravity loads. Many other structures were designed for seismic levels that today are not sufficient, because of the update of the national seismic hazard map. Furthermore, the use of material with lower characteristics than those assumed in the design and their degradation during time yield to even worse seismic performance.

The insertion of BRBs in the RC structure is considered a promising solution for the seismic upgrading of RC buildings, and to this end a design method has been presented in a companion paper. This paper aims at testing the developed design procedure on three case study frames, affected by the most common structural deficiencies of Italian buildings. The design method is applied considering different values of the design parameters, to investigate their influence on the response of each upgraded frame. The seismic performance of the rehabilitated frames is assessed by nonlinear dynamic analysis. The results show the effectiveness of the design method, which successfully designs the BRBs for every frame considered, regardless of the specific structural issues. Moreover, information about the most proper values to assign to the ruling parameters is provided.

Keywords: retrofitting, existing structure, Buckling Restrained Brace

1. Introduction

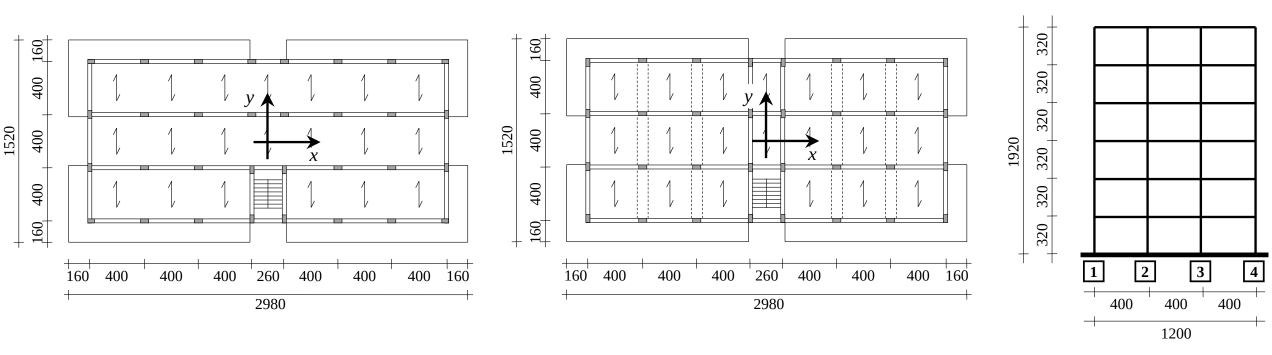
In many of the earthquake prone areas of the world, existing structures often do not fulfill the requirements of seismic regulations. For instance in Italy, an important part of the building heritage is extremely vulnerable to seismic actions, due to different reasons. Firstly, several structures still fully in use date back to the sixties or seventies, when seismic regulations were not in force yet and only gravity loads were considered for the design. Moreover, during the last century the Italian seismic hazard map has been updated several times. As a consequence, new regions were included as seismic areas, or regions already considered seismically prone were classified with an even larger level of hazard. Furthermore, it is not rare that the materials used for construction have lower mechanical properties than those assumed in design, and their natural decay in time has reduced even more the strength and the ductility of the members. Based on these observations such structures should be retrofitted by adding the stiffness and strength necessary to sustain a proper level of seismic forces.

Among the different techniques available for the seismic retrofitting of RC structures, the insertion of Buckling Restrained Braces (BRBs) is considered a promising one. In order to make sure that the upgraded frames exhibit a proper seismic response, the geometrical and mechanical characteristics of BRBs have to be designed accurately. To this end, the authors have developed a design procedure, presented in a companion paper [1]. According to this design method, BRBs are designed to fulfill stiffness and strength requirements to achieve the target limit state. In this paper, this design procedure is validated by retrofitting three RC frames, which are affected by the most common structural deficiencies observed in many Italian buildings. The first case study is a RC frame designed to sustain gravity loads only. This frame presents lateral stiffness and strength, which are inadequate to sustain horizontal forces. Furthermore, due to the irregular distribution along the height of the ratio of demanded shear force over lateral strength, the damage tends to concentrate and promote a story collapse mechanism. The second case study derives from the first one, but the compressive strength of the concrete is assumed lower. Thus, the structural members of this frame have a lower ductility. The third case is a RC frame designed according to the old seismic codes and for seismic levels prescribed for low seismicity areas. This frame shows a more distributed collapse mechanism, but its seismic resistance is still inadequate to overcome strong earthquakes.

In order to assess the seismic response of the frames at the Near Collapse (NC) limit state, incremental nonlinear dynamic analysis is conducted. Afterwards, the three RC frames are upgraded by means of BRBs. The retrofit design is performed several times assuming different values of the ruling parameters, i.e. the behavior factor q and the design story drift Δ_d. Hence, numerical analyses are carried out to determine the seismic performance of the upgraded frames and to compare it to that of the bare frames. From the obtained results, the effectiveness of the design method and the influence of the design parameters are investigated.

2. Design of the examined frames

The analyzed frames were designed to be representative of RC structures with the typical structural deficiencies affecting many of the Italian existing buildings. To this end, two types of RC frames are considered. The first



(a) (b) (c)

Fig. 1 – (a) Plan view of the GL buildings; (b) Plan view of the SR building; (c) Analyzed frame

type, referred to here as gravity load resistant frame (GR frame), is representative of the Italian buildings designed to sustain gravity loads only. The second type, referred to here as seismic resistant frame (SR frame), exemplifies the existing buildings designed according to old Italian seismic standards for low seismicity areas. Fig. 1 shows the plan configurations of the buildings from which the considered frames are drawn. In both cases, the original buildings are six-story high and present four frames with seven bays along the *x*-direction. In the building designed for gravity loads only (Fig. 1a), four frames are arranged along the *y*-direction: two are located on the outermost sides and have three bays, the other two are located next to the staircase and have only one bay. Instead, in the building designed according to old seismic regulations (Fig. 1b), the deck is sustained in the *y*-direction by eight three-bay frames. 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 the features are shown in Fig. 1c.

2.1 Design of the frames for gravity loads only

In order to design the cross-sections of beams and columns of the GL frame, the regulations in force during the seventies in Italy [1]-[4] are applied. The dimensions of the cross-sections are listed in Table 1. The design internal forces of the structural members are determined considering only gravity loads. Dead and live gravity loads are determined considering the nominal values given in [5]. Cross-section size and steel reinforcement of beams and columns are determined by the allowable stress method as stipulated in [6]. For beams, the minimum reinforcement ratio of the tension zone prescribed in [6] is equal to 0.0015. Columns are designed to resist compressive axial force only, while the bending moment is neglected. The design axial force of the column N is evaluated according to the tributary area concept. According to the aforementioned regulations, the minimum required cross-sectional area of the column $A_{c,req}$ is calculated as follows:

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

where $\overline{\sigma}_c$ 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 cylinder strength f_{ck} equal to 20 MPa); steel grade Feb38K with characteristic yield strength 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. Furthermore, rebar area of columns A_s has to be not smaller than the minimum value

$$A_{s,\min} = \max\begin{cases} 0.003\,A_c \\ 0.006\,A_{c,nec} \end{cases} \tag{2}$$

where A_c is the actual cross-sectional area of concrete of the column. Rebars with diameter of 8 mm are used for stirrups. Spacing of stirrups is 150 mm for columns and 200 mm for beams. A more detailed description of the frame GL may be found in [7].

2.2 Design of the seismic resistant frame

The cross-sections of beams and columns of the SR frame are sized considering also the effect of seismic forces. The lateral force method of analysis is applied and the total design seismic force F_h is determined as a function 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 [5] for residential buildings with RC structure:

$$F_h = C\,R\,W \tag{3}$$

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.

Table 1 – Cross-sections of the frame members

GR frame			
Storey	**Columns**		**Beams**
	1 and 4	**2 and 3**	
6th	30x30	30x30	30x60
5th	30x30	30x30	30x60
4th	30x30	30x30	30x60
3rd	35x30	30x40	30x60
2nd	40x30	30x50	30x60
1st	50x30	30x60	30x60

SR frame			
Storey	**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

The analyzed frame has the same geometrical scheme of the frame GL (Fig. 1c), but the cross-section sizes of the structural members are different, and they are listed in Table 1. The considered frame is designed to sustain one fourth of the total seismic force, because 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 [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 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.

3. Evaluation of the frames seismic performance

In order to simulate different levels of seismic deficiencies, three case study frames were derived from the two types of frames designed in Section 2. The first two cases are obtained from the GL frame, while the third case is derived from the SR frame. With regard to the two cases derived from the GL frame, the first one (GL1 frame) is a RC frame totally consistent with the GL frame presented in Section 2 in terms of mechanical properties of materials, loads and seismic weight, size of structural members, etc. The second case study frame (GL2 frame) differs from the GL1 frame for the compressive strength of concrete, which is assumed lower than that considered in design (f_{ck} is equal to 12 MPa instead of 20 MPa). The third case study is obtained from the SR frame assuming that the concrete used for its construction has compressive strength lower than that considered in design (f_{ck} is equal to 12 MPa instead of 20 Mpa). Furthermore, it is assumed that the actual seismic weight of the SR frame is increased by 25% due to modifications of non-structural elements and type of occupancy of the building occurred after its construction.

In order to investigate the need for seismic upgrading of the three frames, the seismic available capacity of each frame is determined and compared to the minimum capacity required by the Italian National Annex [8]. The capacity provided by each frame is evaluated as the maximum PGA the frame can sustain before exceeding the target limit state. To this end, a numerical model of the considered frames is built in OpenSEES [9] to conduct Incremental nonlinear Dynamic Analysis (IDA). The minimum capacity is defined by the Italian National Annex to EC8 as the PGA required for the verification of the target limit state. For the assessment of the considered frames, the target limit state is assumed as the NC limit state, corresponding to the 5% of probabilities of exceedance in 50 years and a required capacity of 0.45 g.

3.1 Numerical model

In order to evaluate the nonlinear response of the case study frames, a two dimensional numerical model has been developed in OpenSEES [9]. In this model, the masses are lumped at the floor levels and all the nodes belonging to the same floor are constrained to have the same horizontal displacement, to simulate the rigid diaphragm effect of the concrete deck. In order to take into account the P-Δ effects a leaning column is included in the numerical model; the gravity load applied on the leaning column is equal to the weight of the numerical model minus that applied directly to the RC frame.

Table 2 – Characteristics of materials adopted for the dynamic analysis

Concrete	GL1	GL2	SR
Cylinder compressive strength (MPa)	28	20	20
Young's modulus (MPa)	29960	27085	27085
Strain at maximum strength	2×10^{-3}	2×10^{-3}	2×10^{-3}
Tensile strength in tension (MPa)	2.77	2.21	2.21

Rebars	GL1	GL2	SR
Yielding strength (MPa)	400	400	450
Young's modulus (MPa)	210000	210000	210000
Ultimate strain in tension	7.5×10^{-5}	7.5×10^{-5}	7.5×10^{-5}
Strain-hardening ratio	0.0066	0.0066	0.0058

The "Beam With Hinges Element" implemented in OpenSEES has been adopted for columns and beams, to simulate elastic members with plastic hinges at their ends. The length of the plastic hinge is 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 is assigned to concrete fibers. An elasto-plastic material with strain kinematic hardening constitutive law is assigned to steel fibers. The parameters used for materials are summarized in Table 2. The area, the moment of inertia of concrete cross-section and the Young's modulus of concrete are assigned to the elastic element. Furthermore, a "ZeroLength Element" is added at one end of each beam. This element is characterized by (i) a small axial stiffness and (ii) a large shear and flexural stiffness. Because of the first feature, the beams can deform axially and the arising of axial force in beams is prevented. However, the second feature ensures the transfer of bending moment and shear force from the beams to the frame node.

In case of the upgraded frame, BRBs are included in the numerical model as truss elements with the cross-sectional area equal to the equivalent area $A_{B,eq}$. This latter is determined by the design procedure introduced in the companion paper [1]. The cyclic behavior is simulated by the material model proposed by Zona and Dall'Asta [10] 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.

Further details about the characteristics adopted in the numerical model can be found in the companion paper [1] and in [7] .

3.2 Assessment of the case study frames by nonlinear dynamic analysis

The Incremental nonlinear Dynamic Analysis has been conducted to evaluate the seismic performance of the analyzed frames. The numerical model defined in Section 3.1 has been 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 assumed as seismic input. The SIMQKE computer program [11] is used to generate these 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 [12]. Five seismic excitation levels are considered to perform the IDA and the values of PGA range from 0.05 g to 0.45 g with a step of 0.10 g.

The seismic performance of the analyzed frames is evaluated in terms of heightwise distribution of the ratio of maximum story drift demand Δ to capacity Δ_{LS}. The drift capacity Δ_{LS} is defined as the story drift angle corresponding to the achievement of the target limit state, which is assumed here equal to the NC limit state. The provisions of EC8 [13] quantify the capacity in terms of chord rotation. For the NC limit state, EC8 defines the chord rotation capacity θ_{um} as the chord rotation at yield plus plastic rotation at column failure θ_{um}^{pl}. Thus, the drift angle capacity Δ_{LS} corresponding to the NC limit state is evaluated by the following equation:

$$\Delta_{LS} = \theta_{um} \frac{L_{cl}}{H} \text{ for NC limit state} \tag{4}$$

where L_{cl} is the clear length of columns and H is the inter-story height of the frame. The drift angle Δ_{LS} is evaluated for the two ends of all the columns of the story and the minimum value is assumed as drift capacity of the story.

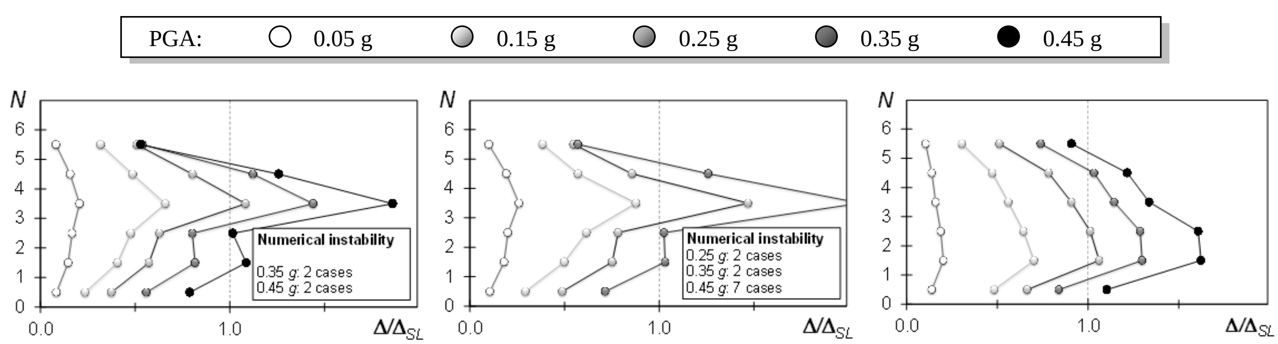
For each of the five considered seismic excitation levels, the drift demand Δ and drift capacity Δ_{LS} are determined at each time step of nonlinear dynamic analysis. Then, the maximum ratio Δ/Δ_{LS} over the duration of the ground motion is evaluated and the median value of Δ/Δ_{LS} over the ten accelerograms is determined at each story. The maximum value of Δ/Δ_{LS} along the height is assumed representative of the seismic performance of the frame. When Δ/Δ_{LS} is larger than 1 the demand overcomes the capacity and the frame exceeds the NC limit state.

Fig. 2a, 2b and 2c show the heightwise distribution of Δ/Δ_{LS} for the GL1, GL2 and SR frame, respectively. For each frame, the different curves show the results of the nonlinear dynamic analysis run at a different seismic excitation level. The NC limit state is exceeded for values of PGA close to 0.25 *g* for the frames GL1 and SR (Fig 2a and 2c, respectively), and just larger than 0.15 *g* for the frame GL2 (Fig 2b). The frame SR (Fig 2c) exhibits the best seismic behavior, being the story drift ratio (i) distributed rather uniformly along the height and (ii) always smaller than that of the frames GL1 and GL2, independently of the seismic excitation level. Both the frames GL1 and GL2 suffer from drift concentration at the fourth story, which becomes more significant when PGA increases (Fig 2a and 2b). Furthermore, for large PGA values, some dynamic analyses of frames GL1 and GL2 terminated prematurely because of numerical instability, which can be identified with collapse in occurrence of the related accelerograms.

In conclusion, none of the three RC frames fulfills the requirements of EC8 for NC limit state and all the frames need to be retrofitted. Moreover, the analysis of the results in terms of drift demand to capacity ratio and number of accelerogram for which numerical instability occurred, shows different levels of seismic deficiency for the three frames: from the frame SR, which is the stiffest and strongest, to the frame GL2, which is the weakest, the most flexible and also less ductile.

4. Design of the seismic upgrading and evaluation of the upgraded frames performance

The three frames analyzed in Section 3 are upgraded by inserting BRBs in the first and third bay of each floor. The axial stiffness and the yield strength of BRBs have been determined according to the design method developed by the authors in the companion paper [1]. According to this design procedure, three requirements have to be fulfilled: (i) the drift demand Δ must be lower than the design story drift Δ_d; (ii) the ductility demand of BRBs must be lower than their ductility capacity $\mu_{B,LS}$; (iii) the total lateral strength V_{Rd}, provided by BRBs and the 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 Δ_d. The first one rules the minimum lateral strength level required to the structure, while the second one is assumed as a fraction of the maximum drift that the bare RC frame can accommodate and control the required stiffness of the frame.



(a) (b) (c)

Fig. 2 – Drift demand to capacity ratio for NC limit state of the frames (a) GL1, (b) GL2 and (c) SR

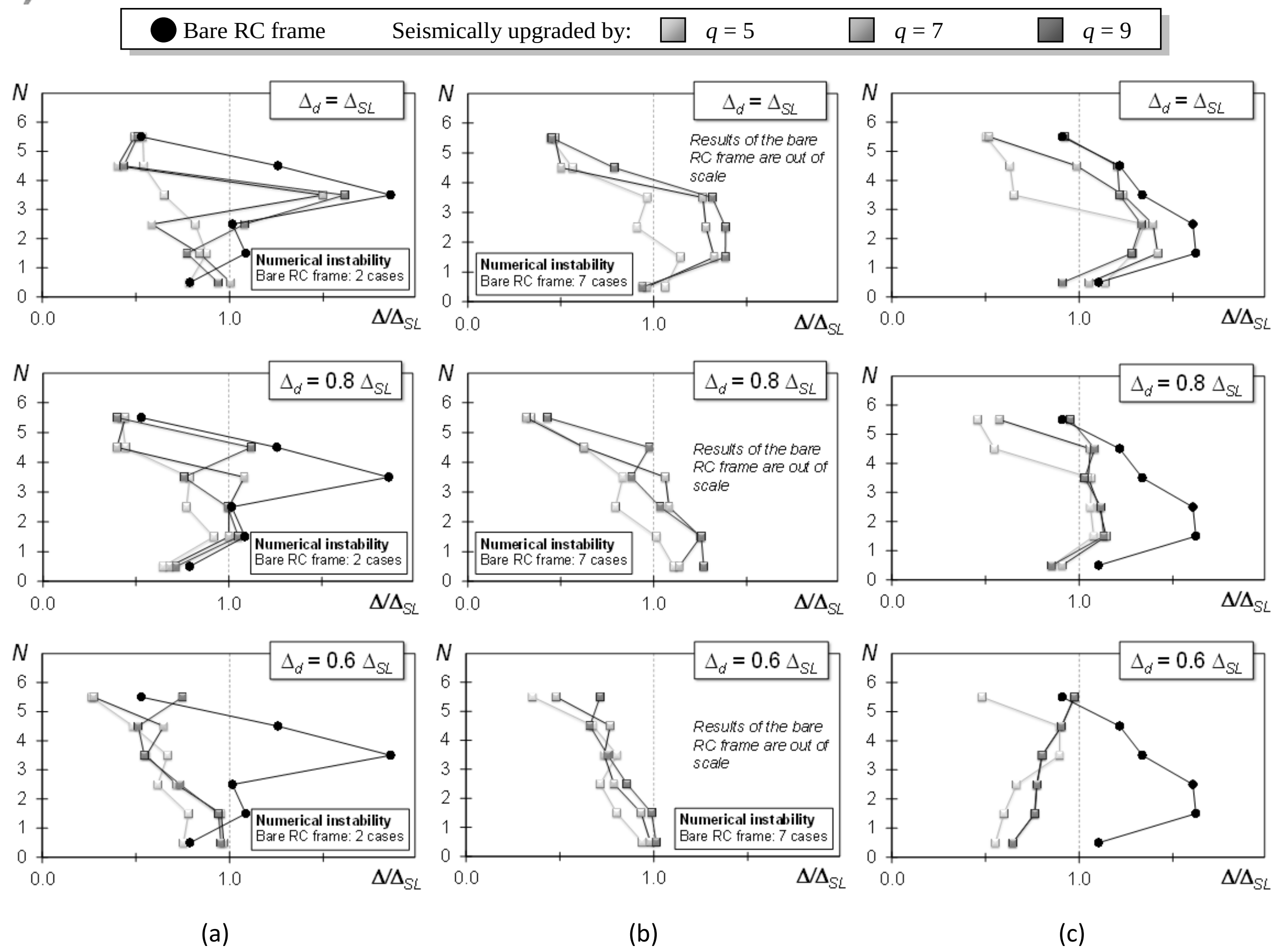


Fig. 3 – Effect of seismic upgrading on drift demand to capacity ratio for NC limit state: (a) frame GL1 (b) frame GL2 and (c) frame SR

In order to investigate the influence of the design parameters on the seismic response of the upgraded frames, the design method has been applied considering three values of design story drift Δ_d, i.e. 1.0, 0.8 and 0.6 times the drift capacity Δ_{LS} of the RC frame. For each value of Δ_d the behavior factor q ranges from 5 to 9 in steps of 2. Afterwards, the performance of the frames upgraded with different values of design parameters has been evaluated by nonlinear dynamic analysis and compared to that of the bare RC frame. For nonlinear dynamic analysis, the set of ten artificial accelerograms defined in Section 3.1 is used and it is scaled by the reference PGA for soil type A equal to 0.45 g. The maximum response is determined at each story for the 10 ground motions and the median value is calculated and represented in following figures.

Fig. 3 shows the results of the nonlinear dynamic analysis in terms of heightwise distribution of the ratio of drift demand Δ over drift capacity Δ_{LS}. The maximum value of the ratio Δ/Δ_{LS} along the height represents the seismic performance of the frame and it is larger than 1 when frames do not match the target limit state. For each value of Δ_d assumed in the design (1.0, 0.8 and 0.6 of Δ_{LS}), the seismic response of the bare RC frame (black circle points) is compared to that of the frames designed with different values of q (white, grey and black squared points). In particular, Fig. 3a, 3b and 3c show the values of Δ/Δ_{LS} demanded at each story of GL1, GL2 and SR frame, respectively. It is notable that the insertion of BRBs improves the seismic response of all the considered RC frames. In fact, the bare RC frame suffered from a severe damage concentration at the 4th story and exhibited numerical instability for two accelerograms. When BRBs are inserted into the RC frames, the concentration of drift demand is reduced and its distribution becomes more uniform along the height. Furthermore, no numerical instabilities occurred during the analysis of any of the three frames. The reduction of drift demands depends primarily on the value assumed for the drift design Δ_d. If large values of Δ_d are assumed,

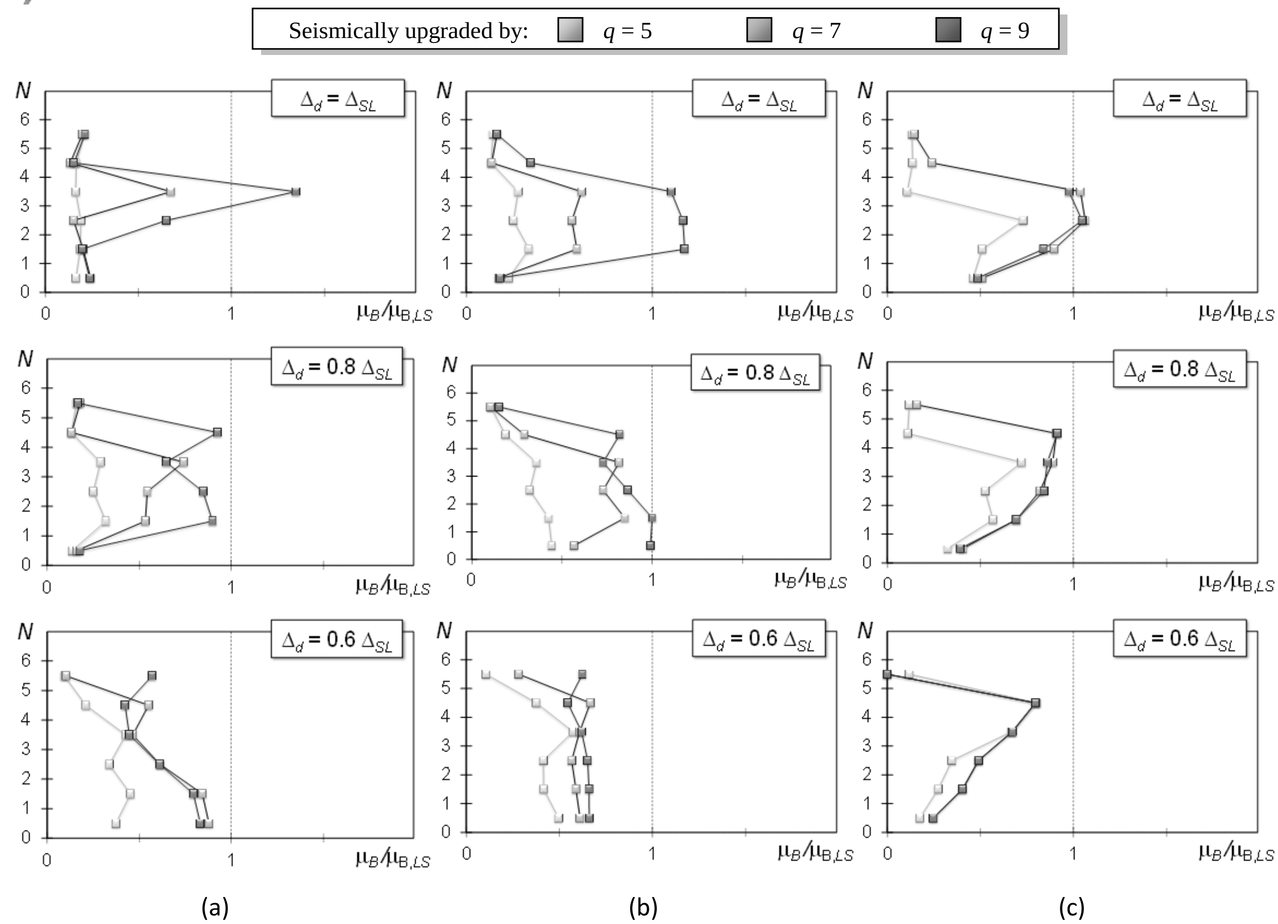


Fig. 4 – Verification of ductility demand of BRBs for NC limit state: (a) frame GL1 (b) frame GL2 and (c) frame SR

such as 1.0 or 0.8 times the drift capacity Δ_{LS}, the response of the upgraded frames generally improves compared to that of the bare frame. However, in all the three frames the ratio Δ/Δ_{LS} still overcomes 1.0. This means that any of the upgraded frames meets the target seismic performance yet, independently of he value of behavior factor q. When Δ_d is assumed equal to 0.6 Δ_{LS} for the BRBs design, both the GL frames and the SR frame upgraded with different values of q satisfy the requirements of the NC limit state verification. Moreover, the final response of all the three upgraded frames does not show a significant dependency on the value assumed for the behavior factor q.

The seismic performance of the frames upgraded with different values of Δ_d and q is also checked in terms of ductility demand of BRBs. Fig. 4a, 4b and 4c show the ductility demand to capacity ratio of BRBs at each story of the GL1, GL2 and SR frame, respectively. The ductility capacity of BRBs $\mu_{B,LS}$ is assumed equal to 25 [14]. It is shown that the ductility requirement is less restrictive than the drift requirement (Fig. 2) and more influenced by the value of behavior factor q. If q is assumed not too large in the BRBs design, such as 5 or 7, the ductility demand of BRBs is always smaller than the capacity, even for $\Delta_d = \Delta_{LS}$. This result occurs in all the three retrofitted frames (Fig. 4a, 4b and 4c). When the drift requirement becomes more restrictive and BRBs are designed with $\Delta_d = 0.6\ \Delta_{LS}$, the ductility requirement is fulfilled in the GL1, GL 2 and SR frame regardless of the value of behavior factor q.

These results show that the optimal seismic performance of the RC frames is achieved when the design method assumes a restrictive requirement on the design story drift, i.e. Δ_d is taken as a smaller percentage of the drift capacity Δ_{LS}. Most notably, despite the three considered frames suffered from different deficiencies and showed different mechanisms, the BRBs designed with $\Delta_d = 0.6\ \Delta_{LS}$ provided every frame with a proper stiffness and strength. The drift demand was reduced below the capacity and a more favorable collapse mechanism was induced.

5. Conclusions

This paper shows the application of a design method of BRBs for the seismic upgrading of RC frames. In particular, this investigation takes into consideration three RC frames, which were designed to include the typical structural deficiencies affecting many existing Italian buildings. The first case study was designed to sustain gravity loads only and suffered from severe concentration of damage and irregular distribution of the shear demand with respect to the shear resistance. The second case study shared the same features of the first one, but the compressive resistance of the concrete was assumed lower than the designed value. The third case study was a frame designed according to old seismic provisions; this frame showed a better seismic response, but however its seismic resistance was still insufficient to face strong ground motions.

The seismic response of the three considered frames at the NC limit state was determined by means of incremental nonlinear dynamic analysis. The results showed that none of the three frames fulfilled the requirement of the considered limit state. Hence, the procedure to design BRBs has been applied considering different values of the design parameters, i.e. the design story drift Δ_d and the behavior factor q. The performance of the upgraded frames has been compared to that of the bare frames at the NC limit state by means of nonlinear dynamic analysis. The seismic response was considered in terms of distribution along the height of (i) ratio of drift demand over drift capacity and (ii) ratio of ductility demand over ductility capacity of BRBs.

Based on the results presented in this paper, the proposed design method provided the BRBs with the proper value of stiffness and strength. In particular, the drift requirement resulted to be more restrictive than the ductility requirement and it influenced the efficiency of the retrofitted method. In fact, when the design method was applied with $\Delta_d = 0.6\ \Delta_{LS}$ both the drift requirement and the ductility requirement were fulfilled at each story in each frame, regardless of the behavior factor adopted. Most notably, the effectiveness of the design method was not influenced by the structural deficiencies of the different frames. Despite the structural inadequacies that affected the three frames were different, the design method with $\Delta_d = 0.6\ \Delta_{LS}$ yielded to the fulfillment of all the requirements, mitigated the damage concentration and led to a more dissipative collapse mechanism in all the frames.

6. References

[1] Barbagallo F, Ghersi A, Marino EM, Stramondo PR, (2017) A design procedure for seismic upgrading of existing r.c. frams by BRBs, 16th World Conference of Earthquake Engineering, Santiago Chile, Chile.

[2] Italian Ministry of Public Works: Law n. 1086, 5/11/1971, Norme per la disciplina delle opere in conglomerato cementizio normale e precompresso ed a struttura metallica (Regulations for constructions of normal and pre-stressed reinforced concrete and with steel structure), *Gazzetta Ufficiale Serie generale n. 321*, 21/12/1971, Rome. (in Italian)

[3] Italian Ministry of Public Works: Ministry Decree, 30/05/1974, Norme tecniche per la esecuzione delle opere in cemento armato normale e precompresso e per le strutture metalliche (Technical regulations for constructions with reinforced concrete, pre-stressed concrete and steel structure), *Gazzetta Ufficiale Serie generale*, 29/07/1974, Rome. (in Italian)

[4] Royal Decree n. 2229, 16/11/1939, Norme per l'esecuzione delle opere in conglomerato cementizio semplice ed armato (Regulations for constructions of concrete and reinforced concrete). *Supplemento Gazzetta Ufficiale n. 92*, 18/04/1940, Rome. (in Italian)

[5] Italian Ministry of Public Works: Ministry Decree, 16/01/1996, Norme tecniche per le costruzioni in zone sismiche (Regulations for constructions in seismic areas), *Gazzetta Ufficiale Serie generale*, 5/02/1996, Rome. (in Italian)

[6] Italian Ministry of Public Works: Ministry Decree, 14/02/1992, Norme tecniche per l’esecuzione delle opere in cemento armato normale e precompresso e per le strutture metalliche (Technical regulations for constructions with reinforced concrete, pre-stressed concrete and steel structure), *Suppl. Ord. Gazzetta Ufficiale*, 18/03/1992, Rome. (in Italian)

[7] Stramondo PR. (2015) Formulazione di un metodo di progetto per l’adeguamento sismico di telai in c.a. mediante controventi a instabilità impedita (Formulation of a design method for seismic upgrading of RC frames by buckling restrained braces). Ph.D. Dissertation, University of Catania, Italy. (in Italian)

[8] Italian Ministry of Infrastructure and Transport: Ministry Decree, 21/07/2012, Approvazione delle Appendici nazionali recanti i parametri tecnici per l'applicazione degli Eurocodici (Ratification of National Annexes with parameters ascribed for application of Eurocodes), *Supplemento Gazzetta Uffi ciale n. 73, 27 marzo 2013 - Serie generale*, Rome. (in Italian)

[9] Mazzoni S, McKenna F, Scott MH, Fenves GL, Jeremic B (2003) OpenSEES command language manual. Berkeley: Pacific Earthquake Engineering Research Center, University of California.

[10] Zona A, Dall'Asta A (2012) Elastoplastic model for steel buckling-restrained braces, *Journal of Constructional Steel Research*; **68**: 118–125

[11] SIMQKE. (1976) A program for artificial motion generation, User's manual and documentation, Department of Civil Engineering MIT.

[12] Amara F, Bosco M, Marino EM, Rossi PP (2014) An Accurate Strength Amplification Factor for the Design of SDOF Systems with P-Δ Effects. *Earthquake Engineering and Structural Dynamics*; **43**: 589–611.

[13] CEN. Eurocode 8: Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings, EN 1998–3. European Committee for Standardization: Bruxelles, Belgium, 2005.

[14] Bosco M, Marino EM, Rossi PP (2015) Design of steel frames equipped with BRBs in the framework of Eurocode 8. *Journal of Constructional Steel Research*; **113**: 43–57.