Inelastic behavior of base-isolated RC frame buildings

D. Cardone, A. Flora & G. Gesualdi
University of Basilicata, Italy

SUMMARY
The results of a parametric study on the inelastic seismic response of base isolated RC frame buildings are presented. Buildings characterized by number of storeys ranging from 2 to 8, strength ratios ranging from 0.03 to 0.15 and post-yield stiffness ratio ranging from 0% to 6% have been examined, also taking into account the effects of infilled masonry panels. The selected buildings show a structural configuration typical of many existing RC buildings, realized in Italy and other European countries in the '60s and '70s. The study is based on the results of extensive nonlinear response-time history analyses, using a set of seven seismic ground motions. Different types of isolation systems have been considered, including: HDRB, LRB and FPB. The results are expressed in terms of global ductility demand to the superstructure as a function of the strength reduction factor imposed to the superstructure with respect to its elastic behavior.

Keywords: Seismic Isolation, RC Frames, Inelastic behavior, Nonlinear response time-history analysis

1. INTRODUCTION

The seismic isolation technique (Naeim and Kelly, 1999) being based on the reduction of the seismic effects on the structure rather than on structural strengthening, appears to be an appealing strategy for the seismic retrofit of existing buildings. Several numerical and experimental studies have definitely demonstrated the applicability and potentials of base isolation for the seismic protection of buildings designed for gravity loads only or with substandard seismic details. More importantly, throughout the world, there are several examples of application of seismic isolation for the retrofit of existing (including historical) buildings (Kelly, 1998; Mokha *et al*, 1996). Nevertheless, in many cases, seismic isolation turns out to be inapplicable for existing buildings, at least based on the usual philosophy and current practice of use of the seismic isolation technique. Usually, indeed, the performance objective in the design of buildings with seismic isolation is to maintain the structure in the linear elastic range even under strong earthquakes. Existing buildings, designed for gravity loads only or according to old seismic codes can exhibit global strength ratio (α = max base shear divided by the weight of the superstructure) as low as 2-5 % (Kunnath *et al*, 1995). In this case, fundamental periods of vibration of the base-isolated building as large as 5-6 sec may be needed to guarantee the elastic response of the superstructure. Very large isolation periods result in very large horizontal displacements, which may be not compatible with the width of the available separation joints or even not compatible with the displacement capacity of the currently used isolation device.
Recent studies by Faccioli and Paolucci (2004) indeed, have shown that deriving maximum displacements from their spectral acceleration counterpart, through the well-known relationship $S_d = S_a/\omega^2$, leads to underestimate the maximum displacements of structures with period of vibration greater than the corner period (T_D in the Eurocode 8 (2003)) between the constant-velocity and constant-displacement segments of the response spectrum. Such studies, indeed, demonstrated that T_D tends to increase with the earthquake magnitude and it is also affected by the epicentral distance of the site.
All that considered, it is apparent that, in most of the cases, problems may arise using both elastomeric isolators (instability under large shear displacements) and friction pendulum systems (poor re-centring

capacity for low-rise buildings; excessive vertical displacements for high-rise buildings). In any case, larger horizontal displacements result in larger expansion joints and more difficult technical solutions for staircases, elevator, lifelines, etc. In other words, larger horizontal displacements can make less attractive and more expensive the seismic retrofit of the building. An alternative approach could be that of reducing the isolation period (and as a consequence the maximum displacement of the isolation system) while accepting limited plastic deformations in the superstructure (under strong earthquakes with long return period), thus avoiding any strengthening measure in the superstructure. From this point of view, it is interesting to examine the inelastic behavior of building with seismic isolation.
Only a few studies have examined the behavior of buildings with seismic isolation featuring superstructures that undergo significant plastic deformations during the design basis earthquake (Ordonez *et al*, 2003; Aiken *et al*, 2008). This situation is representative of what would happen if the seismic isolation is adopted for the seismic upgrading, rather than the seismic retrofit, of existing buildings. The studies conducted so far, considering an elasto-perfectly plastic cyclic behavior of the superstructure, pointed out that the ductility demands to the superstructure may become significant and non-linearly dependent on the superstructure strength when subjected to different seismic ground motions (Ordonez *et al*, 2003).
In this paper, the results of a comprehensive parametric study on the inelastic seismic response of buildings with seismic isolation are presented. The final goal of this study is to assess the applicability and effectiveness of seismic isolation for the seismic upgrading of existing buildings. To this aim, particular attention will be paid to: (i) the conditions under which it is favourable, for the safety and the costs of the intervention, the use of seismic isolation (ii) the most suitable type of isolation system and (iii) the minimum level of strength of the superstructure required to avoid excessive ductility demands or brittle collapse.
The parametric study described in this paper is based on nonlinear response-time history analyses of a number of RC building prototypes, using a set of seven artificial and natural seismic ground motions. At this stage of the study, reference to a two degrees of freedom (2-DOF) model has been made to establish a general understanding of the inelastic response of seismically isolated structures. the results of the analyses are presented in terms of global ductility demand to the superstructure as a function of the strength reduction imposed to the superstructure with respect to its elastic response (i.e. a sort of behavior factor for base-isolated buildings).

2. CASE STUDIES

2.1. Building prototypes

The building prototype selected in this study for numerical analyses is a typical multi-storey RC frame building realized in Italy before 1975, when the first Italian seismic code enforced. It is designed for gravity loads only and presents substantially a symmetric structural layout in both horizontal directions (see Fig. 1) and regularity characteristics in elevation, with number of storeys typically ranging from 2 to 8.The structure features internal resistant frames in one direction only (i.e. orthogonally to the floor deck spans, see Fig. 1), identified as the strong direction of the building. In the orthogonal weak direction, the structure features two perimetric resistant frames only. The infilled masonry panels of the perimetric frames present large openings in the strong direction whilst no openings in the weak direction (see Fig. 1). For that reason, the effects of infilled masonry panels can be deemed to be significant in the weak direction only.
The selected building prototype presents two spans along the short side and five spans along the long side of the building. An average cylindrical compressive strength equal to 22 MPa and a yield strength equal to 375MPa have been assumed for concrete and steel reinforcement, respectively, in line with the typical mechanical properties of the materials of pre-75 RC buildings.
A simulated structural design has been carried out to determine steel reinforcement of beams and columns, based on the internal forces computed on the basis of the characteristic values of dead and live loads (2.0 kN/m^2 for residential buildings). The size of beam sections has been taken equal to 300×500mm at all the storeys while that of the columns have been taken equal to 300×300mm for the outer columns and ranging from 300×300mm to 300×900mm (depending on the number of storeys of

the building) for the inner columns. The effective fundamental period (T_e) of the buildings was found to increase almost linearly with the number of storeys, being of the order of 0.44 sec (0.5 sec) in the strong (weak) direction of 2-storeys buildings while some 1.6 sec (1.9 sec) in the strong (weak) direction of 8–storeys buildings. These values are in good accordance with those found by other authors in previous studies (Masi and Vona, 2008).

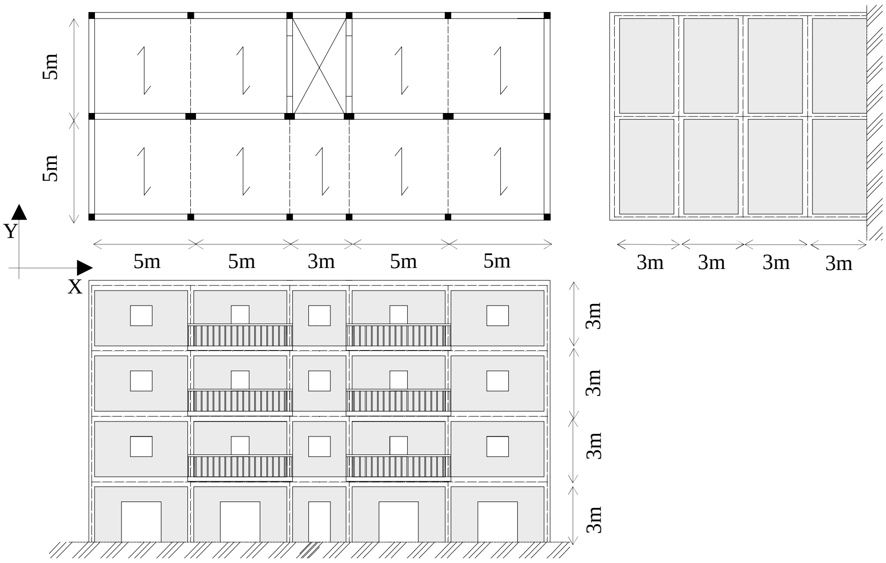

Figure 1. Typical layout of the examined RC frame buildings

2.2. Lateral force-displacement behavior of the superstructure

The lateral force-displacement skeleton curves of the building prototypes considered in this study have been derived from pushover analysis, carried out with the structural analysis program SAP2000_Nonlinear (2004). A concentrated plasticity model has been adopted in the pushover analysis of the buildings. The infilled masonry panels have been modeled with compression-only elastic fragile struts, with axial stiffness equal to 40000 KN/m and axial strength equal to 140 KN. The aforesaid values have been computed referring to well-known modeling assumptions for infilled masonry panels (Fardis, 1996), assuming a width/length ratio of the equivalent strut equal to 0.1, according to (Mainstone, 1974), and considering the lowest between the ultimate strengths associated to shear, sliding and compression collapse mechanisms (Zarnic and Tomazevic, 1985). The pushover curves (see Fig. 2) have been replaced by idealized bilinear relationships, in accordance with FEMA356 2000. Strength ratios ranging from approximately 3% to 15% and post-yield stiffness ratio (r_s=K_2/K_1) ranging from approximately 0% to 6% have been found for the selected prototypes w/o infills.

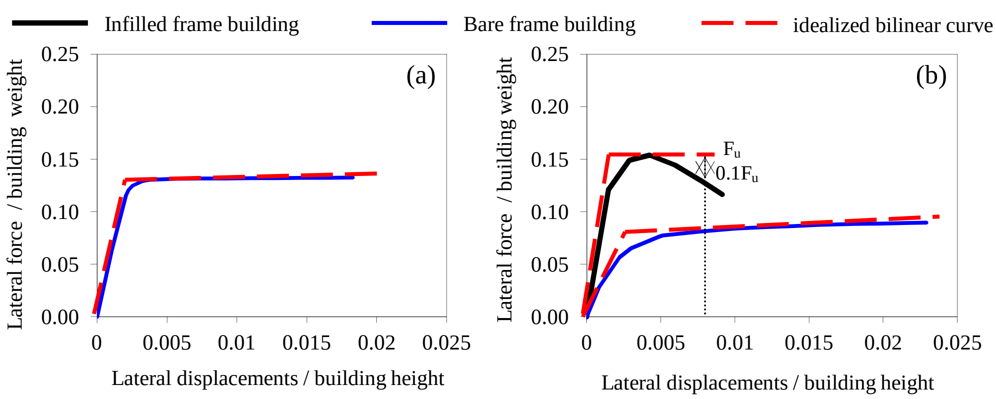

Figure 2. Typical pushover curves of the examined RC frame buildings in their (a) strong and (b) weak direction

2.3. Parameters of the analysis

In the parametric analyses, 72 models of superstructure have been considering, differing in number of storeys (N_s = 2, 4, 6 and 8), direction of analysis (weak and strong), strength ratio of the structure ($\alpha = F_y/W$= 0.05, 0.10 and 0.15), and post-yield stiffness ratio (r_s = 0%, 3% and 6%). Moreover, two different seismic intensity levels have been considered, equal to 0.35g (≈1.2*1.2*0.25g) and 0.5g (≈1.2*1.2*0.35g), respectively. The aforesaid PGA values correspond to the design earthquake levels on type B soil with a probability of exceedance of approximately 5% in 50 years (return period of approximately 975 years) in moderate and high seismicity regions, respectively.
Three different types of isolation systems have been examined, namely: High Damping Rubber Bearings (HDRB), Lead Rubber Bearings (LRB) and Friction Pendulum Bearings (FPB). The main design parameters of each type of isolation system (i.e. the viscous damping ratio for HDRB, the post-yield hardening ratio and lead ductility ratio for LRB, the friction coefficient for FPB) have been selected in such a way to cover typical situations that can be find in the current practice. In particular, three viscous damping ratios have been assumed for HDRB, equal to 10%, 15% and 20%, respectively. A post-yield hardening ratio of 12% and three different lead ductility ratios, corresponding to equivalent viscous damping of 15%, 20% and 25%, respectively, have been considered for LRB. Finally, two different friction coefficients (2% and 4%, precisely) have been assumed for FPB.
First, the isolation systems have been designed for a force level compatible with the yield strength of the superstructure ($F_{max,el} \approx F_y$). During the nonlinear response-time history analyses, the yield strength of the superstructure has been progressively reduced by a strength reduction factor ($\beta = F_y/F_{max,el}$), typically varied between 1 and 0.6 (with step 0.05-0.1), in order to simulate the entry of the structure in the plastic range. Two different approaches have been followed. In the first approach (see Fig. 3(a)), the strength of the superstructure has been reduced while keeping the effective initial stiffness of the superstructure unchanged. In the second approach (see Fig. 3(b)), the strength of the superstructure has been progressively reduced, while keeping the yield displacement of the superstructure unchanged.

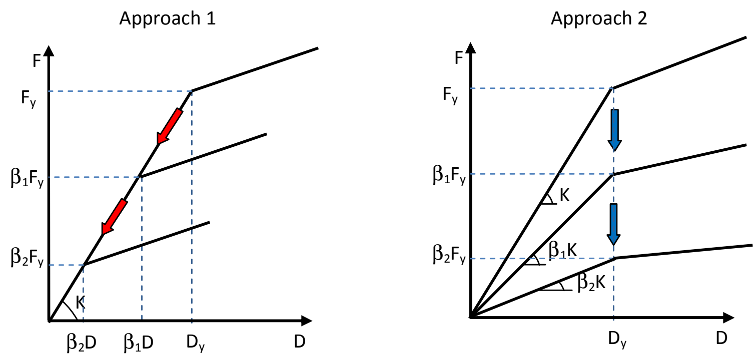

Figure 3. Different approaches followed in the reduction of the lateral strength of the superstructure

3. NUMERICAL MODELING

The base isolated buildings have been modeled in SAP2000 as 2-DOF systems with lumped masses equal to m and N_s*m, respectively, where m is the floor mass. The first DOF corresponds to the displacement of the isolation system, the second DOF to the displacement of the superstructure. The floor mass m has been computed referring to a residential building with gross floor area of approximately 230m^2, resulting equal to about 300 ton. The equivalent viscous damping of the superstructure has been assumed equal to 3%. The isolation system and the superstructure have been modeled as nonlinear springs using the NLLink elements of SAP2000.

3.1. Isolation systems

The cyclic behavior of LRB (see Fig. 4(a)) has been described by an elasto-plastic with hardening model using the Plastic-Wen NLLink element of SAP2000. According to Naeim and Kelly (1999), the cyclic behavior of HDRB (see Fig. 4(a)) has been described as a combination of a linear elastic model, a pure hysteretic model (energy dissipation proportional to D) and a pure viscous model (energy dissipation proportional to D^2), using the Linear, Plastic-Wen and Damper NLLink elements of SAP2000, respectively. The cyclic behavior of FPS (see Fig. 4(b)), finally, has been described as a combination of a linear elastic model and a rigid-perfectly plastic model using the Linear and Plastic-Wen NLLink elements of SAP2000, respectively.

3.2. Superstructure

The cyclic behavior of the superstructure has been described by means of a "Thin" Takeda degrading-stiffness-hysteretic model (see Fig. 4(c)). The Multilinear Plastic Pivot NLLink finite element of SAP2000 has been used, assuming a = 1.5 and b = 0.3, where a and b are the constitutive parameters of the model which locate the pivot point for unloading/reloading to/from zero from/towards a given cyclic force, respectively. More detailed information on the Multilinear Plastic Pivot finite element can be found in Dowell *et al.* (1998). The cyclic behavior of the superstructure after seismic strengthening, shown in Fig. 5(a) has been described with a "Fat" Takeda degrading-stiffness hysteretic model, assuming a = 3.7 and b = 0.6 as constitutive parameters of the Multilinear Plastic Pivot NLLink element of SAP2000.

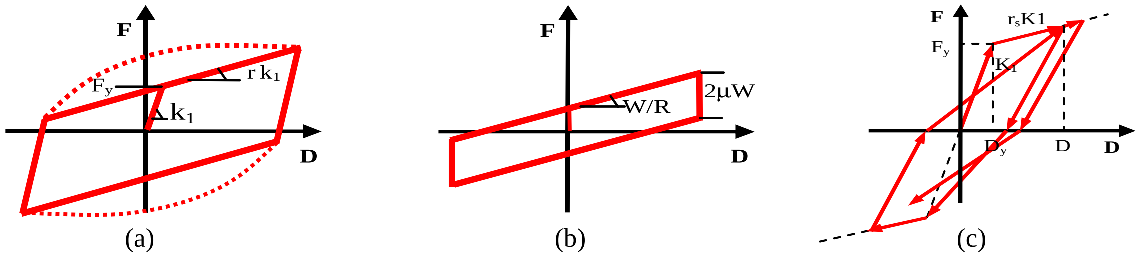

Figure 4. Idealized cyclic behavior of (a) LRB-HDRB, (b) FPS and (c) RC building.

3.3. Seismic ground motions

A set of seven accelerograms, compatible (on average) with the EC8 response spectrum for soil type B, has been used in the nonlinear response-time history analyses. The set includes three artificial/synthetic ground motions and four Italian natural records. The four natural records have been scaled to fit the EC8_soil B reference spectrum (same average spectral velocity,i.e. same area below the acceleration spectra) in the period range of interest for base-isolated buildings (1 sec to 3.5 sec).
During the analyses, the accelerograms have been alternatively scaled at 0.35g and 0.5g. The selected PGA values are compatible with those provided by the current seismic codes for the verification of the collapse limit state of structures located on stiff soil (soil type B according to EC8) in medium (a_g = 0.25g) to high (a_g = 0.35g) seismicity regions.

4. RESULTS

Figure 5 compares three different strategies for the seismic protection of an existing 4-storey building with α = 5%, subjected to an earthquake design intensity of 0.5g, with a probability of exceedance of 5% in 75-100 years. The first strategy is based on the increase of the lateral strength of the structure by 4 times (from 5% to 20%), which leads the global ductility demand of the fixed-base structure to acceptable values (e.g. around 4, as shown in the example of Fig. 5(a)). The second strategy is based on the adoption of an isolation system with high equivalent viscous damping (ξ_{eq}≈25%), designed to prevent yielding in the superstructure. This results in a base-isolated building with fundamental period of vibration of approximately 6 second maximum displacement as large as 460mm (see Fig. 5(b)). The third strategy is based on the use of a stiffer isolation system that limits the elongation of the period of vibration of the base-isolated building to approximately 3 sec, corresponding to a maximum base displacement of approximately 100mm and a global ductility demand to the superstructure of the order of 2 (see Fig. 5(c)), which can be deemed to be compatible with the ductility capacity of existing RC buildings (Calvi, 2008).

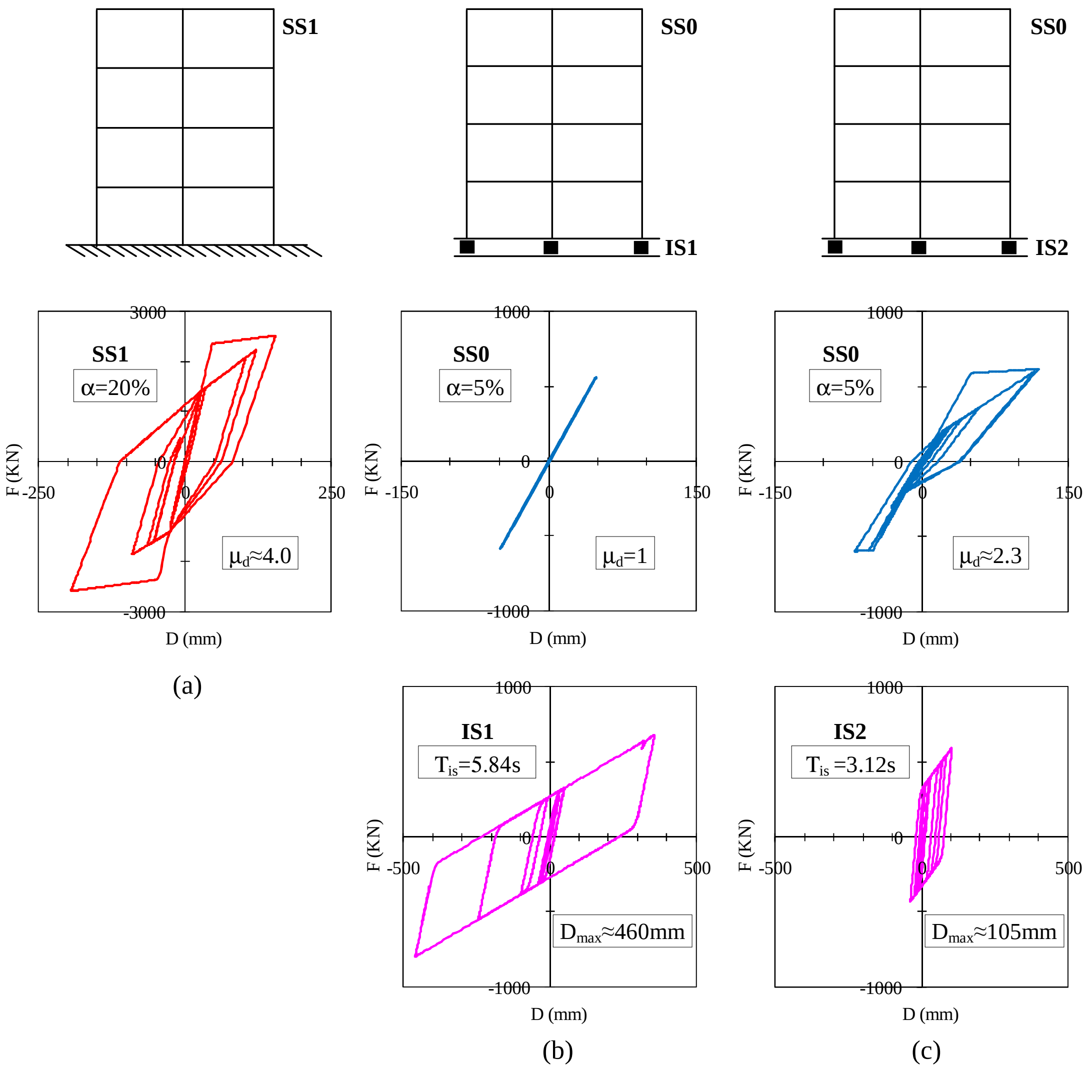

Figure 5. Different strategies in the seismic protection of a 4-storeys building with α=5%: (a) lateral strengthening , (b) use of an isolation system (ξ≈25%) designed to prevent yielding in the superstructure, (c) use of an isolation system designed to limit the maximum base displacement while accepting a ductility demand of 2 in the superstructure.

Figure 6 compares the cyclic hysteretic behavior of the superstructure of a fixed-base 2-storey building (α = 20%) with the cyclic hysteretic behavior of the superstructure of a base-isolated 2-storey building (βα = 3%) experiencing similar ductility demands (around 7) under the same seismic ground motion (SMQ-1 at 0.35g). As can be seen, the cyclic hysteretic behavior of a base-isolated structure is quite different from that of a fixed-base structure. The fixed-base structure, indeed, experiences several large inelastic cycles, which result in a considerable amount of energy dissipated during the earthquake. The seismic response of the structure with seismic isolation, on the contrary, is characterized by a few inelastic cycles (just one in the case considered in Fig. 6(b)), which result in a small amount of energy dissipated during the seismic event.

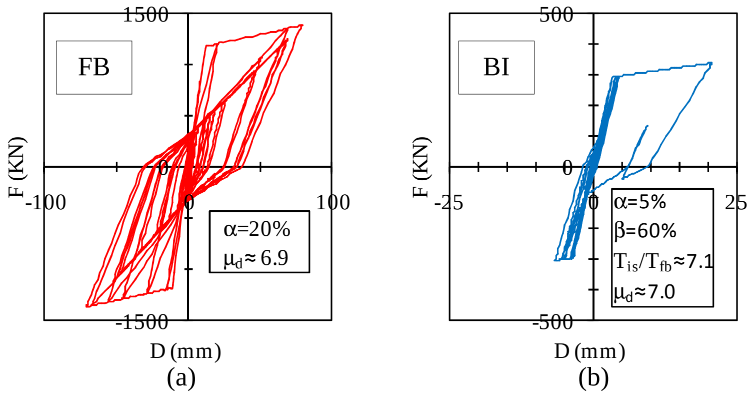

Figure 6. Comparison between the cyclic hysteretic behaviors of the superstructures of 2-storey (a) fixed-base and(b) base-isolated buildings experiencing similar ductility demands.

Nevertheless, the energy dissipation capacity of a fixed-base structure turns out to be considerably greater than that of a base-isolated structure experiencing similar ductility demands under the same seismic ground motion (see Fig. 7). In any case the energy dissipated by the isolation system results considerably greater than that dissipated by the superstructure through its hysteretic cyclic behavior, even when the isolation ratio is quite low. As a consequence, even in presence of significant ductility demands to the superstructure, the energy dissipation capacity of a base-isolated building is mainly dominated by the energy dissipation capacity of the isolation system. As a result, the inelastic behavior of the superstructure little affects the seismic response of the isolation system, especially for low-rise buildings (see Fig. 8).

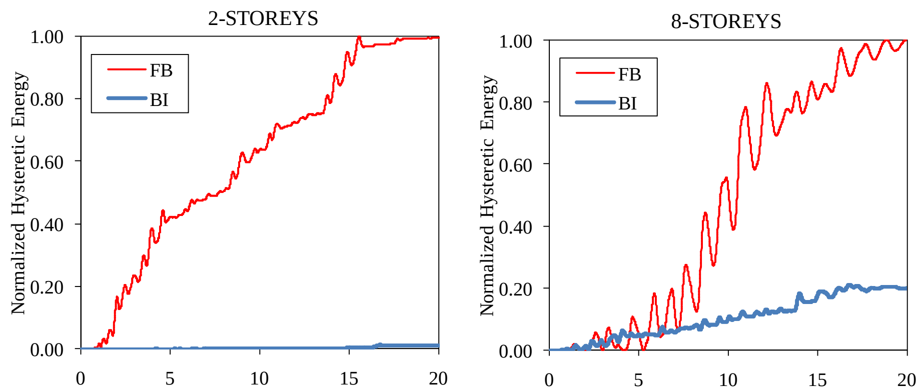

Figure 7. Comparison between the hysteretic energy dissipated by the superstructures of Fixed-Base (FB) and Base-Isolated (BI) buildings experiencing similar ductility demands under the same earthquake.

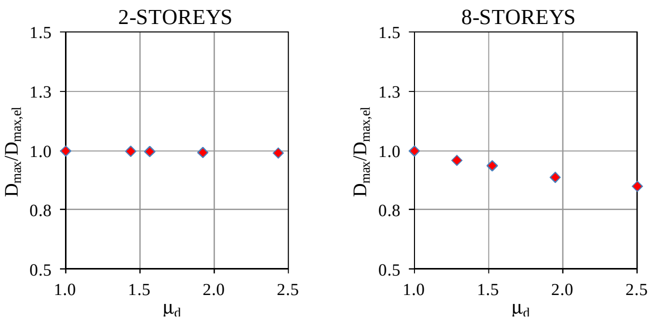

Figure 8. Changes in the maximum displacement response of the isolation system due to the hysteretic cyclic behavior of the superstructure, for different values of ductility demand

Figures 9 shows the average ductility demands to the superstructure (α = 10%) of base-isolated buildings as a function of the strength reduction factor (β) imposed to the superstructure (following the first approach of Fig. 3) at 0.5g PGA. The global ductility demands reported in Figs. 10 have been obtained by averaging the NTHA results over the selected seven seismic ground motions and two horizontal directions of analysis (see Fig. 1). Figures 9 points out that the global ductility demand of base-isolated buildings strongly increases while decreasing the number of storeys of the building, mainly due to the reduction of the yielding displacement of the superstructure, as a consequence of its higher lateral stiffness.

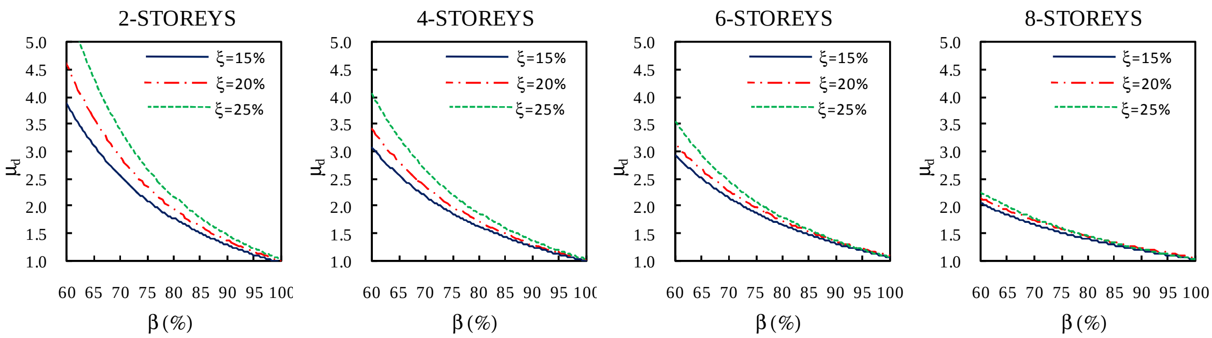

Figure 9. Average ductility demand (μ_d) as a function of the strength reduction factor (β) imposed to the superstructure (α=10%, r_s=3%) of 2-, 4-, 6- and 8-storeys buildings equipped with LRB at 0.5g PGA

As shown in Fig. 10, the global ductility demand slightly reduces while increasing the strength ratio of the superstructure (on average by 12% passing from α = 5% to α = 15), as well as the post-yield stiffness ratio of the superstructure (on average by 6% passing from r_s = 0% to r_s = 6%).

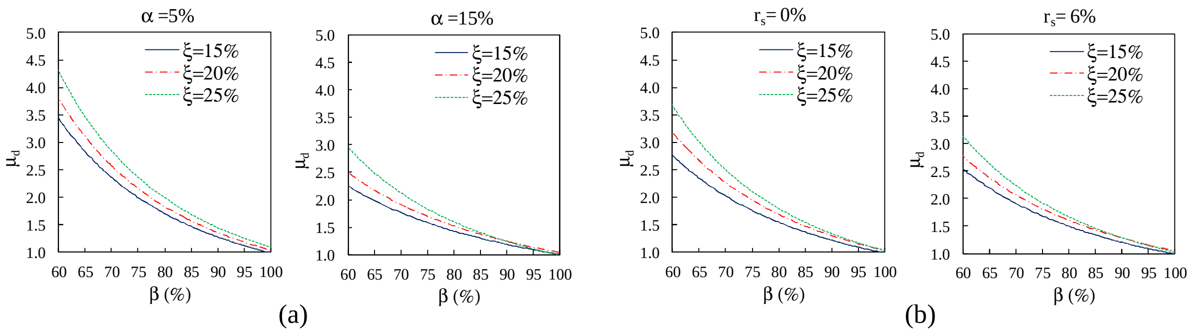

Figure 10. Effects of (a) strength ratio (α) and (b) post-yield stiffness ratio (r_s) on the global ductility demand to the superstructure (μ_d) of a 4-storeysbase-isolated (LRB) building, at 0.35g PGA

Figures 11 and 12 emphasize the effects of masonry infills on the inelastic response of the selected building prototypes in their weak direction. Figure 11, in particular, refers to a 4-storey building (α = 10%) equipped with different types of isolation systems, i.e.: (a) LRB with $\xi_{eq} \approx 20\%$, (b) HDRB with $\xi_{eq} \approx 15\%$ and (c) FPB with $\xi_{eq} \approx 10\%$, respectively.
As can be seen, the presence of strong infills w/o openings, effectively bonded to the RC frame along the entire perimeter and uniformly distributed along the height of the building, can significantly increase the lateral strength of the building, thus reducing the ductility demands to the superstructure for a given seismic intensity. The effectiveness of the masonry infills in enhancing the lateral strength of the frame structure, hence reducing the global ductility demand to the structure for a given seismic intensity, strongly depends on the lateral deformability of the structure, being higher for low-rise building (Fig. 12(a)) while resulting practically negligible for high-rise buildings (Fig. 12(d)).

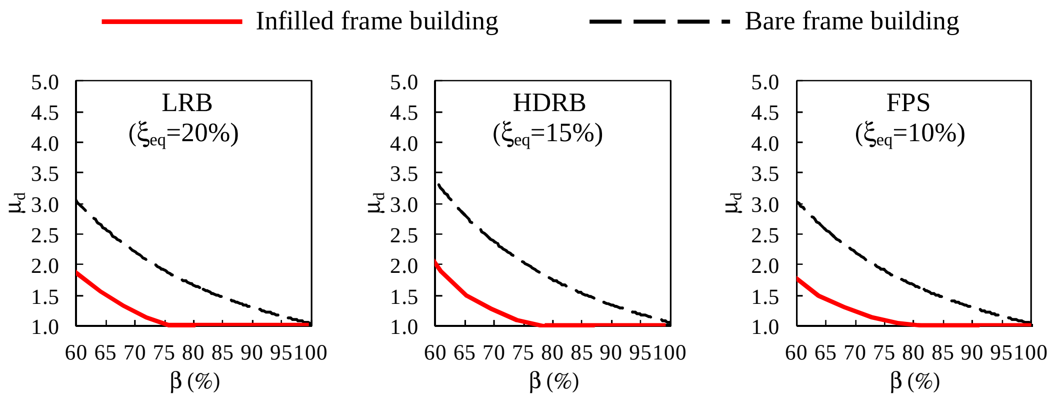

Figure 11. Effects of masonry infills on the ductility demand to the superstructure of a 4-storeys building (α = 10%), equipped with different types of isolation system at 0.5g PGA

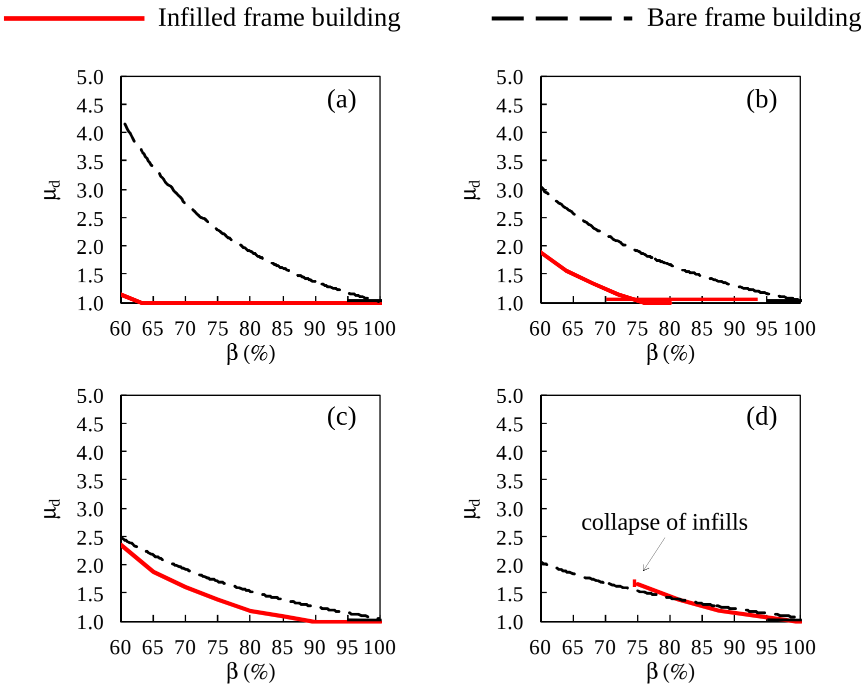

Figure 12. Effects of masonry infills on the global ductility demand to the superstructure of (a) 2-, (b) 4-, (c) 6- and (d) 8-storeys BI-buildings equipped with LRB ($\xi_{eq} \approx 20\%$), at 0.5g PGA.

5. CONCLUSIONS

The results of a parametric study on the inelastic seismic response of base isolated buildings have been presented. The results of this study indicate that the inelastic behavior of a structure with seismic isolation is quite different from that of the same fixed-base structure, especially for two remarkable aspects. Damage in a fixed-base structure results in significant energy dissipation and, as the structure is damaged, its effective period of vibration moves away from the dominant period of vibration of the ground motion, limiting the force demand to the structure. On the contrary, a structure with seismic isolation experiences fewer inelastic cycles and in any case its energy dissipation capacity is dominated by that of the isolation system. As a result, the inelastic behavior of the superstructure little affects the maximum response of the isolation system, especially for low-rise buildings. This suggests that, although limited plastic deformations can be accepted, the collapse limit state of seismically isolated structures should be based on the lateral capacity of the superstructure without significant reliance on its inherent hysteretic damping or ductility capacity.

Obviously, more studies are needed to definitely assess the applicability and effectiveness of seismic isolation for the seismic upgrading of existing RC frame buildings, paying particular attention to the conditions under which it is favourable, for the safety and the costs of the intervention, the use of seismic isolation as alternative strategy to structural strengthening.
Upcoming studies shall be conducted on refined three-dimensional numerical models of RC frame buildings, in which the cyclic behavior of the potential plastic hinges of all the structural members (beams, columns, walls) are individually modelled, taking into account degrading cyclic effects and considering the shear resistance of each structural member. This should give a full understanding of the minimum level of strength of the superstructure required to avoid excessive ductility demands or brittle collapse in the structural members of the superstructure.
Based on the preliminary results of the parametric study presented in this paper, strength reductions (with respect to the minimum level of force necessary for the elastic response of the superstructure) of the order of 25-30% for low-rise buildings and of the order of 35-40% for high-rise buildings seem to be acceptable, being associated to a global ductility demand to the superstructure of the order of 2. Further strength reductions may be accepted taking into account the favourable contribution of the masonry infills. Finally, negligible differences in the seismic performances of different types of isolation systems (including HDRB, LRB and FPB) have been observed, which seems to suggest that basically does not exist a more suitable type of isolation system for the seismic upgrading of existing buildings.

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