Nonlinear seismic behavior of an existing RC building retrofitted with BRBs

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SUMMARY
The paper presents the seismic performance assessment of a four-storey reinforced concrete (RC) existing framed structures designed for gravity loads only. The sample structure exhibits high seismic vulnerability, i.e. low lateral resistance and limited translation ductility. The structural assessment and retrofitting was deemed necessary due to a variation of the building occupancy. An extensive experimental test program was carried out to estimate the mechanical properties of the concrete and steel reinforcement in the existing RC building. Natural frequencies and damping ratios are experimentally investigated by operational modal analysis; the estimated modal properties were employed to calibrate the numerical finite element structural model. The rehabilitation scheme utilized for the sample existing is also presented herein and the retrofitting strategy emphasizes the use of Buckling Restrained Axial Dampers (BRADs) that are a type of buckling restrained braces (BRBs) in which the dampers are used in series with traditional capacity-designed steel braces. Such braces can be conveniently installed along the perimeter frames of the multi-storey buildings to lower the seismic demand on the existing structure and regularize its dynamic response. Local strengthening can, however, be necessary. The adopted design approach assumes that the global response of the inelastic framed structure is the sum of the elastic frame (primary system) and the system comprising perimeter diagonal braces (secondary system); the latter braces absorb and dissipate a large amount of hysteretic energy under earthquake ground motions. Comprehensive nonlinear static (pushover) and incremental dynamic (response history) analyses were carried out for both the as-built and retrofitted structures to investigate the efficiency of the adopted intervention strategy. A suite of code-compliant natural strong motion records was selected and employed to perform inelastic response history analyses. The outcomes of the inelastic analyses show that the existing structure exhibit extremely poor strength capacity and energy dissipation. It is also demonstrated that, under moderate and high magnitude earthquakes, the damage experienced by the retrofitted structural system is concentrated in the added dampers and the response of the existing RC framed structure is chiefly elastic.

Keywords: BRBs, existing RC building, nonlinear static analyses, nonlinear dynamic analyses

1. INTRODUCTION

A large number of existing building structures in the seismic regions of the Mediterranean area was constructed around the early sixties, i.e. they do not implement requirements for the seismic design. For example, in Italy, nearly 60% of the existing reinforced concrete structures were built before the 70s (Manfredi et al., 2007); they are designed primarily for gravity loads only. Such structures possess high seismic vulnerability and should be retrofitted in order to guarantee adequate structural and non-structural performance under earthquake loading. Survey carried out in the aftermath of recent world-wide earthquakes (for example, L'Aquila, Italy, April 2009, Van-Erçis, Turkey, October 2011) emphasized that widespread damage and/or structural collapse occurred for buildings that do not incorporate seismic details (Figure 1). Damage has been observed in RC residential, hospital and school buildings which exhibited low energy absorption and dissipation capacity. Such buildings included primarily low-to-medium rise framed structures designed for gravity loads.

Figure 1 Damage observed in buildings designed for gravity loads in the 2011 Van (Turkey) earthquake.

The most common structural configurations for lateral-resisting systems are concentrically brace frames (CBFs), which possess a lateral stiffness significantly higher than that of unbraced frames, e.g. moment resisting frames. Nevertheless, due to buckling of the metal compression members and material softening due to the Bauschinger effect, the hysteretic behaviour of CBFs with traditional steel braces is unreliable. Alternatively, buckling.restrained braces (BRBs) may be employed as diagonal braces in seismic retrofitting of steel and RC frames designed for gravity loads only. Such braces exhibit compressive strength which is about 10-15% greater than tensile; the global buckling is inhibited (Iwata et al., 2000). Frames with BRBs are being used for new and existing structures worldwide (e.g. Bozorgnia and Bertero, 2004; Di Sarno and Manfredi, 2012, among many others), especially for damage controlled structures as shown pictorially in Figure 2 and initially formulated by Wada et al. (1997).

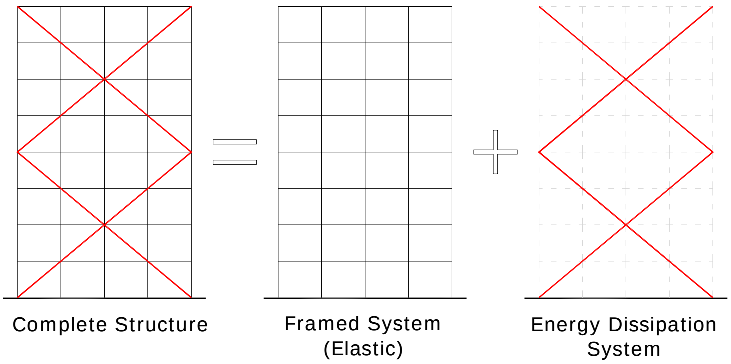

Figure 2. Damage controlled structure.

The global response of the inelastic structural system can be assumed as the sum of the elastic frame (also termed primary structural system) and the system formed by the diagonal braces (secondary system) that absorbs and dissipates large amount of hysteretic energy under earthquake ground motion. The primary system is capable to withstand vertical loads and behaves elastically under earthquake loads. The secondary system includes the dissipative members and is thus designed to damp the seismic lateral actions and deformations. Dissipative members, such as BRBs, may be installed in the exterior frames of multi-storey buildings and can be thus easily replaced in the aftermath of a devastating earthquake. Primary and secondary systems act as a parallel system; the lateral deformation of the structure as a whole corresponds to the deformation of both primary and secondary systems.
The present analytical work discusses the seismic performance assessment of a four-storey reinforced concrete (RC) existing framed structure designed for gravity loads only. The same structure exhibits

high seismic vulnerability, i.e. low lateral resistance and limited translation ductility. The structural assessment and retrofitting was deemed necessary due to a variation of the building occupancy. The retrofitting strategy adopted includes the use of BRBs. Such braces can be conveniently installed along the perimeter frames of the multi-storey buildings to lower the seismic demand on the existing structure and regularize its dynamic response. Nonlinear static (pushover) and dynamic (response history) analyses were carried out for both the as-built and retrofitted structures to investigate the efficiency of the adopted intervention strategy. The outcomes of the inelastic analyses demonstrate that, under moderate and high magnitude earthquakes, the damage experienced by the retrofitted structural system is concentrated in the added dampers and the response of the existing RC framed structure is chiefly elastic.

2. BUILDING DESCRIPTION

2.1. Geometry

The sample RC existing framed building was built in the late 1930s; the structural system is characterized by a rectangular plan layout whose sides are 54.5 m and 18.5 m long and the total height is 19.2 m, as shown in Fig. 3. The structural system consists of frames placed along a single direction only; the stairs are located in a slightly eccentric position, as shown in the Fig. 3.

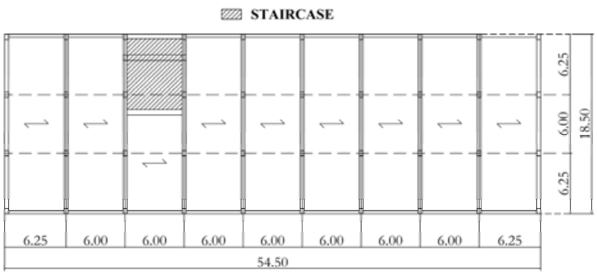

Figure 3. Plan layout of the sample as-built building (dimensions in metres)

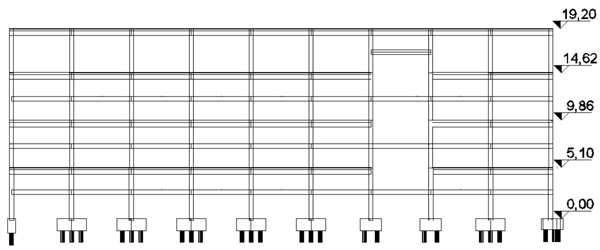

Figure 4. Section layout of the sample as-built building (dimensions in metres)

The 4-storey building has interstorey varying between 4.58 m and 5.10 m, as shown in Fig. 4, and the floors are placed at a height of 5.10 m, 9.86 m, 14.62 and 19.2 m; furthermore, the building is characterized by the presence of beams located at mid-height of the storeys along the perimeter, as shown in Fig. 4. The roof floor is flat and the floor slabs consists of 21 cm and 23 cm deep cast in situ concrete and brick decks at the first floor and all the other floors, respectively. The solid slab thickness is 5 cm at all floors; thus diaphragmatic behavior may be thus assumed. The as-built framed system employs deep foundations consisting of plinths on piles, the piles are connected with tie-beams.
Rectangular cross-section columns and beams are used only; the cross-section properties are summarized in Table 2.1.

Table 2.1. Geometry of the columns and the beams of the sample building (dimensions in metres)

Columns				Beams			
1st floor	2nd floor	3rd floor	4th floor	1st floor	2nd floor	3rd floor	4th floor
0.45x0.50	0.40x0.50	0.40x0.40	0.35x0.35	0.26x0.685	0.26x0.645	0.26x0.68	0.35x0.765
0.45x0.60	0.40x0.55	0.40x0.45	0.40x0.35	0.45x0.40	0.40x0.645	0.40x0.68	0.35x0.81
0.45x0.70	0.40x0.60	0.40x0.50	0.40x0.40	0.45x0.48	0.40x0.80	0.40x0.80	0.35x0.885
0.45x0.75	0.40x0.65			0.45x0.685	0.40x0.85	0.40x0.85	0.35x0.945
				0.45x0.70	0.40x0.90	0.40x0.90	
				0.45x0.75	0.45x0.40	0.45x0.73	
				0.45x0.80	0.45x0.48		
					0.45x0.645		
					0.45x0.70		

The gravity loads (G_k) and the variable loads (Q_k) acting on the sample structure are summarized in Table 2.2.

Table 2.2. Gravity loads (G_k), variable loads (Q_k) and design seismic loads ($G_k+\Psi_E \cdot Qk$) considered

	1st floor	2nd floor	3rd floor	4th floor
G_k (kN/m^2)	5.92	5.91	5.91	5.01
Q_k (kN/m^2)	3.00	3.00	3.00	0.50
$G_k+\Psi_E \cdot Qk$ (kN/m^2)	7,72	7,71	7,71	5,01

2.2. Material properties

A comprehensive experimental test program (in situ and in laboratory) was carried out to estimate the mechanical properties of the concrete and the steel reinforcement in the existing RC building. Additional in situ tests were carried out on the structural system components, i.e. floor slabs and plinths; natural frequencies and damping ratios are experimentally investigated by operational modal analysis; the estimated modal properties were employed to calibrate the numerical finite element structural model. Cylinder concrete samples with diameters of 100 mm were tested under compression to estimate the concrete compression strength f_{cc}. The latter is a function of the diameter and the height of the sample and the cylinder compression strength of the test specimens (BS 1881, Masi 2005). Storey-dependent concrete compression strengths are adopted: fcc_1=19.16 MPa, fcc_2=18.51 MPa, fcc_3=13.44 MPa and fcc_4=22.50 MPa, for the first, the second, the third and fourth floor, respectively. Tensile tests were also carried out on steel reinforcement smooth bars; the laboratory tests showed quite uniform yield strength *fy* and ultimate strength *fu* for the first, the second and the fourth floor and rather lower yield strength for the third floor. Thus, it is assumed values of *fy*=320.38 MPa and *fu*=418.18 MPa for the third floor and values of *fy*=393.96 MPa and *fu*=479.72 MPa for all the other floors. On average, the estimated material overstrength is about 1.24 and the ultimate elongation is higher than 10%, demonstrating a good ductility of the steel reinforcement. Further details on the material properties and the outcomes of the experimental tests are available in Chiodi et al. (2011)

2.3. Structural details

The structural details were investigated either by visual inspection or with pacometric tests. Investigations were carried out for determining the amount and placement of transverse and longitudinal reinforcements. The investigations showed that the structural details of beam-columns do not comply with modern codes of practice for earthquake prone regions. The steel reinforcement comprises smooth bars and the spacing of the transverse stirrups is insufficient to warrant adequate shear resistance to beam, columns and structural joints.

2.4. Structural retrofitting strategy

To evaluate the life cycle costs associated to different rehabilitation schemes that may be utilized for existing RC framed buildings, a set of possible retrofitting strategy has been considered and analyzed. Such intervention schemes can be targeted to achieve different Performance Levels for vulnerable structures in earthquake-prone regions. Emphasis is on the use of BRADs that are a type of BRBs in which the dampers are used in series with traditional capacity-designed steel braces; such braces can be conveniently installed along the perimeter frames of the multi-storey buildings to lower the seismic demand on the existing structure and regularize its dynamic response, as shown in Fig. 5. The selected braces consist of a traditional metallic brace and a buckling restrained device (damper) and exhibit compressive strength greater than tensile strength; thus the buckling is inhibited and a large amount of hysteretic energy can be dissipated. The behavior of the buckling restrained device may be modelled as bilinear response, both in tension and compression. The behavior of the traditional metallic brace may also be schematized as bilinear, except in the case of buckling of brace. The overall brace system comprises two in-series components. The brace resistance is thus equal to the strength of the weaker element, while the deformability of the entire system is equal to the sum of the deformability of the individual elements. The use of BRBs implies also a contribution of additional damping than that of the as-built structure (Kim and Choi, 2004); for the retrofitted schemes a conventional viscous damping coefficient equal to 10% has been assumed.

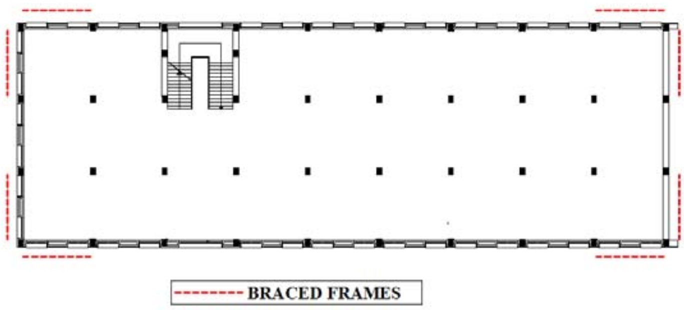

Figure 5. Plan layout of the braced frames

Local strengthening can, however, be necessary. Local strengthening aims at ensuring a ductile behavior of beams, columns and panel joints; it is also effective to augment the shear strength. The confinement also produces an appreciable increase in terms of compressive strength of concrete (see, for example, Mander et al., 1988). Confinement of the ends of the columns has also a beneficial effect with regard to the potential buckling of the longitudinal bars when the spacing of transverse stirrups is insufficient (e.g. Cosenza and Prota, 2006). Local strengthening is carried out either by the use of carbon fiber reinforced polymers or through the use of L press-bent and high-strength steel metal strips (quoted as CAM system). Both are extended to all the nodes as well as beams and columns.
The evaluation of the effects of the local strengthening is carried out in compliance with the provisions implemented in the Italian Technical Code (D.M. 2008).

3. EARTHQUAKE INPUT SELECTION

Seismic hazard has been characterized as the mean annual frequency of exceeding a given level of spectral acceleration at the fundamental period of structure. The hazard values are taken from the tabulated values in INGV, Italian National Institute of Geophysics and Volcanology, in which, the mean annual rate of exceeding an earthquake event of interest has been calculated using probabilistic seismic hazard analysis (PSHA) for the site of the structure. INGV has evaluated probabilistic seismic hazard for each node of a regular 5 km spacing grid that covers the whole Italian territory with over 13000 nodes (Meletti and Montaldo 2007). The results are provided in hazard curves in terms of PGA and spectral acceleration, Sa(T), for ten different periods from 0.1 to 2 s. In Incremental Dynamic Analisys (IDA) approach the seismic motion has been represented in terms of ground acceleration time-histories. An ensemble of natural accelerograms was selected according to Italian Code specification using software REXEL (Iervolino 2010), as summarized in Table 3.1.

Table 3.1. Natural accelerograms selected in the present study

Waveform ID	Earthquake ID	Station ID	Earthquake Name	Date	Mw	PGA_X (m/s^2)	PGA_Y (m/s^2)
0133	63	ST33	Friuli (aftershock)	15/09/1976	6	1.123	0.932
0151	65	ST33	Friuli (aftershock)	15/09/1976	6	0.810	0.884
0335	158	ST121	Alkion	25/02/1981	6.3	1.144	1.176
0386	176	ST152	Lazio Abruzzo (aftershock)	11/05/1984	5.5	0.365	0.331
0600	286	ST223	Umbria Marche	26/09/1997	6	1.685	0.956
0772	350	ST223	Umbria Marche (aftershock)	03/10/1997	5.3	0.423	0.405
6958	2154	ST772	Izmit (aftershock)	31/08/1999	5.1	0.130	0.143

Figure 6 provides the acceleration response spectra of the selected records both for X- and Y-direction.

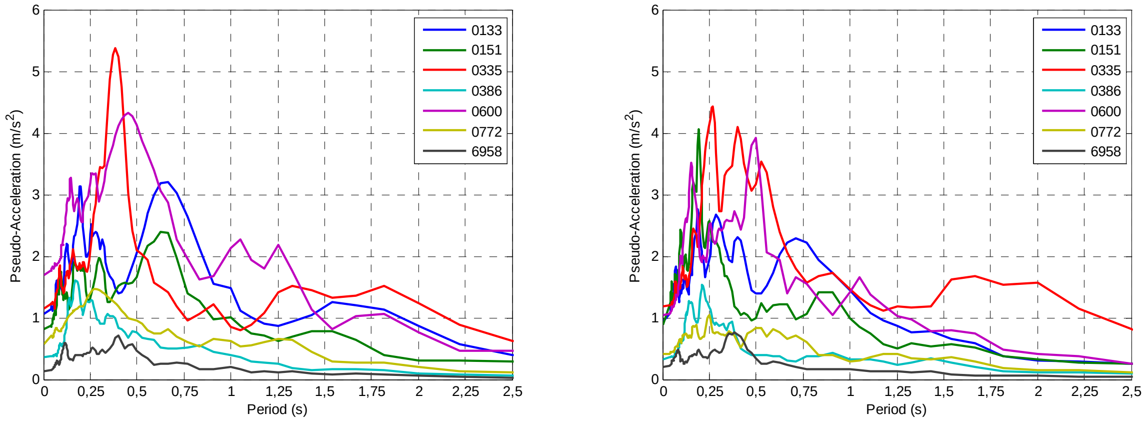

Figure 6. Acceleration response spectra of the selected earthquake records: X-dir. (left) and Y-dir. (right)

The records were scaled from 0.05 g to 0.30 g and in the present study only the results coming from the first five selected accelerograms are presented.

4. STRUCTURAL ASSESSMENT

Accordingly to the Italian Technical Code (D.M. 2008), the information and data available for the geometry, the structural details and the material properties of the sample structures are considered sufficient to adopt the LC3 level of knowledge and consider a confidence factor CF=1.

4.1. The Finite Element Model

Refined three-dimensional (3D) finite element models were employed to analyze the sample framed as-built and retrofitted structures and analyze the earthquake response. Fig. 7 displays the FE model utilized for the assessment of structural response of both the as-built and the retrofitted configuration, elaborated with SAP2000 code, vers. 15.1.0 Advanced (CSI, 2012).

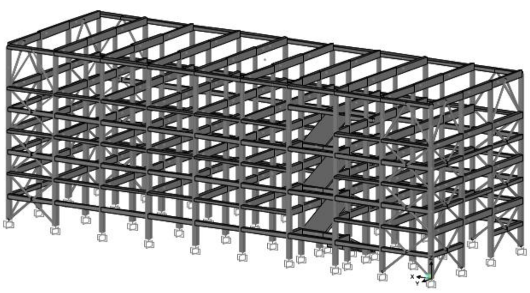

Figure 7. Finite element model for nonlinear response history analyses

4.2. Eigenvalue analyses

The dynamic properties of the as-built and retrofitted structures have been evaluated and the results of the eigenvalue analyses are reported in Table 4.1. The period of vibration of the first six modes and the participation masses along the principal directions of the structure are reported.

Table 4.1. Results of eigenvalue analysis for both as-built and retrofitted structures

Mode	As-built structure				Retrofitted structure			
	Period (s)	MpX (%)	MpY (%)	Mpρz (%)	Period (s)	MpX (%)	MpY (%)	Mpρz (%)
1	1.00	62.3	3.4	27.2	0.75	78.8	0.8	13.8
2	0.87	10.0	29.4	43.7	0.67	1.2	73.6	61.6
3	0.66	0.1	40.7	5.5	0.51	0.4	5.2	6.1
4	0.44	16.7	0.1	2.1	0.28	13.2	0.1	1.7
5	0.35	0.3	8.2	11.1	0.26	0.1	13.4	9.6
6	0.31	0.1	6.8	0.5	0.19	0.0	0.0	2.6
Total		89.5	88.5	90.2		93.7	93.1	95.5

The results demonstrated that the as-built structure was characterized by a fundamental mode of vibration along the longitudinal axis whose period is equal to 1.00 s. The second mode of vibration, whose period is equal to 0.87 s, has a significant torsional coupling due mainly to the difference between the mass centroid and stiffness centroid due to the stairs. In order to calibrate the finite element model and test its reliability with respect to the quality of vibration modes, the natural frequencies and the damping ratios are experimentally investigated by operational modal analysis tests; further details are available in Chiodi et al. (2011).

4.3. Nonlinear static analyses

Two lateral force patterns were employed for the seismic structural assessment:

- a modal pattern, proportional to lateral forces consistent with the mode of vibration determined by the eigenvalue analysis;
- a uniform pattern, based on the distribution of mass along the height.

Fig. 8 and 9 provide the Force Displacement Response Curves and the Acceleration Displacement Response Spectrum (ADRS) of the as-built and retrofitted structures, respectively; they are calculated along the X- and Y- direction, for positive and negative directions of the lateral loadings, with regard to the modal pattern of lateral loads (G1) and the uniform pattern of lateral loads (G2). It is worth mentioning that a conventional viscous damping coefficient equal to 4% and 10% have been assumed for the as-built and retrofitted structures, respectively. The performance points at life safety (LSLS) limit states with respect to both shear and moment beams capacities (SBC and MBC, respectively) and shear and moment columns capacities (SCC and MCC, respectively) are also included. The computed results showed that the as-built system is characterized by a low stiffness and ductility along X-direction, that is the weaker direction due to the lack of frames.

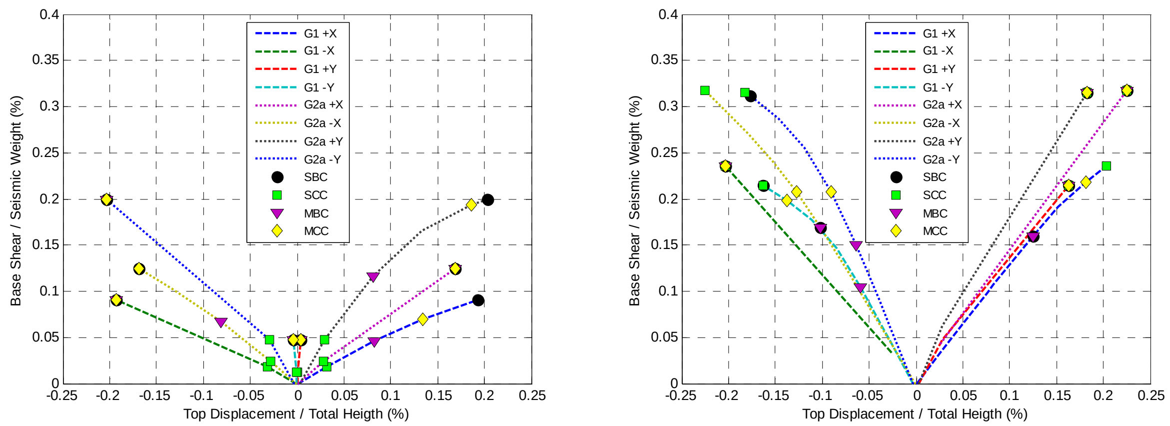

Figure 8. Force Displacement Response Curves of the as-built (left) and retrofitted (right) structure

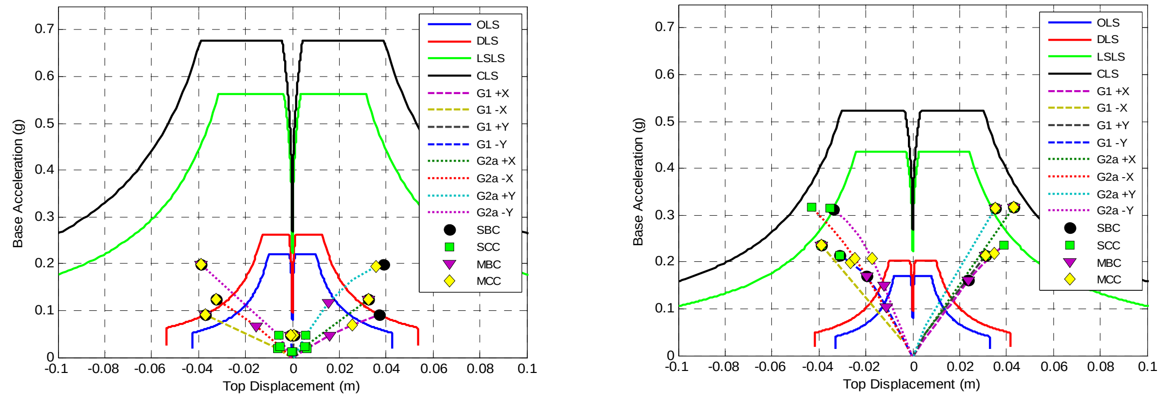

Figure 9. Acceleration Displacement Response Spectrum (ADRS) of the as-built (left) and retrofitted (right) structure

4.4. Nonlinear dynamic analyses

The results provided in Fig.10 prove that the use of BRBs has reduced the scatter of the inter-storey drifts for the retrofitted structures. The computed results also show that the response become regular along both X- and Y-directions.

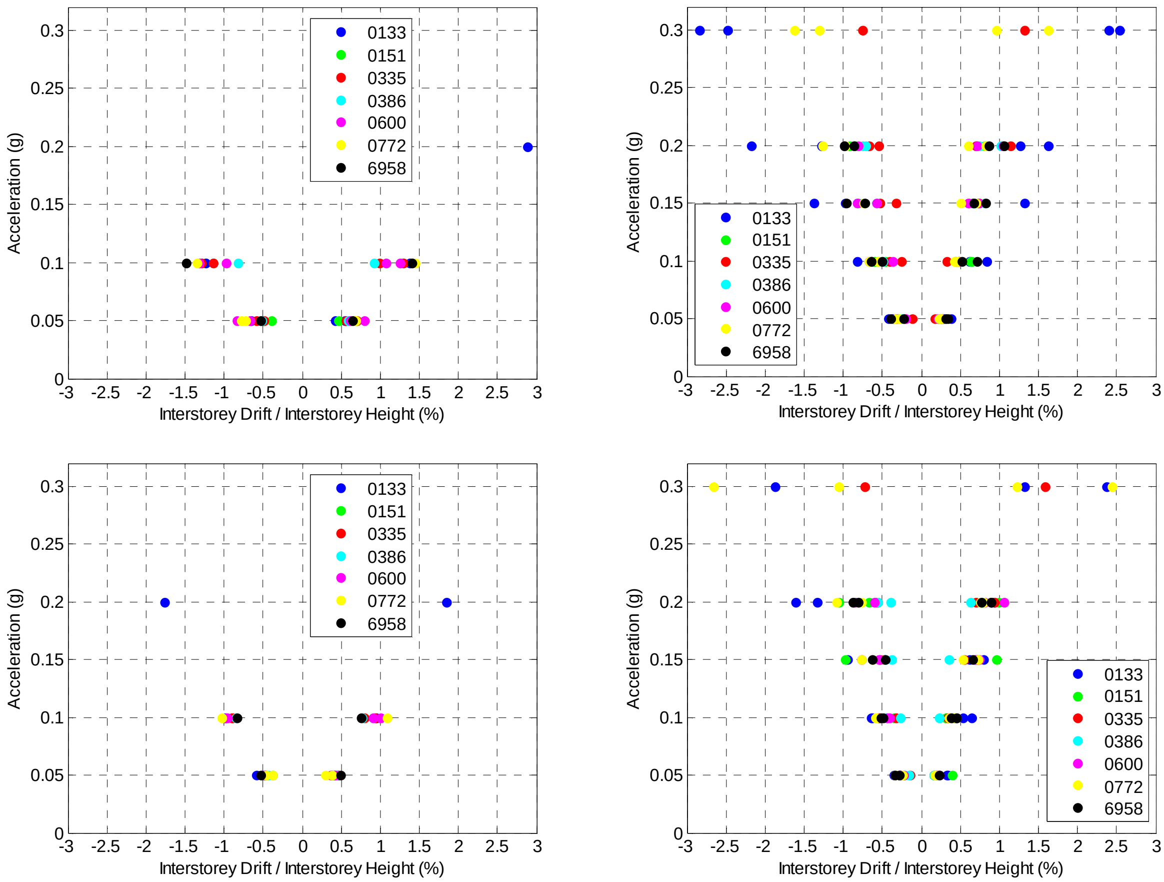

Figure 10. Inter-storey drifts of the as-built (left) and retrofitted (right) structure for the X (top) and Y-direction (bottom)

The results provided in Fig.11 prove that the use of BRBs has reduced also the scatter of the storey shear for the retrofitted structures. The computed results are referred to X-direction.

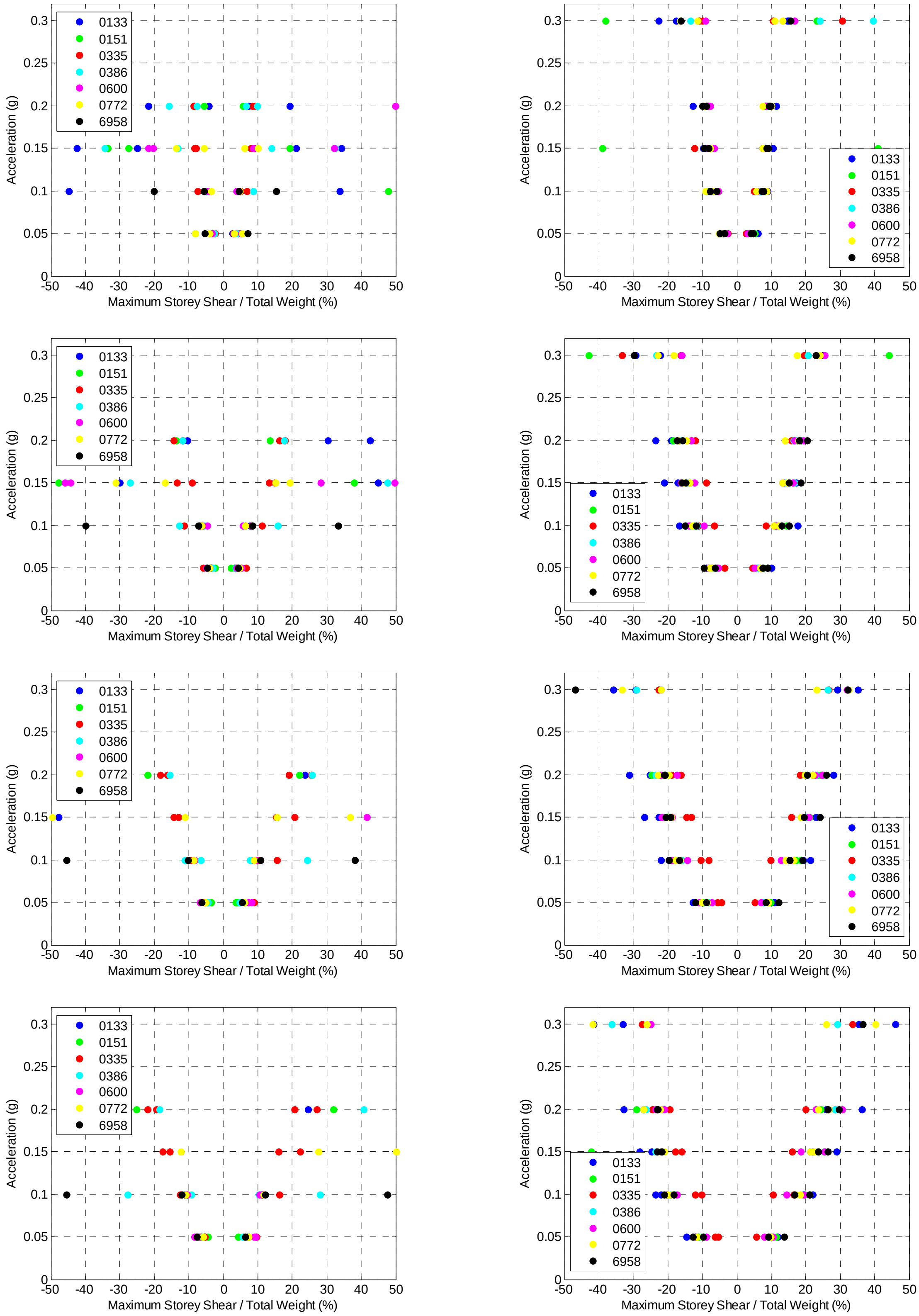

Figure 11. Storey shear response of the as-built (left) and retrofitted (right) structure for the fourth (top), the third, the second and the first floor (bottom)

The behavior of BRBs during earthquakes is still under investigation both in terms of force and displacement time histories and energy response history.

5. CONCLUSIONS

The paper assesses the seismic performance of a reinforced concrete (RC) existing framed structures designed for gravity loads only retrofitted with buckling restrained braces (BRBs). Such braces can be conveniently installed along the perimeter frames of the multi-storey buildings to lower the seismic demand on the existing structure and regularize its dynamic response. Comprehensive nonlinear static (pushover) and incremental dynamic (response history) analyses were carried out for both the as built and retrofitted structures to investigate the efficiency of the adopted intervention strategy. The results show that the existing structure exhibits extremely poor strength capacity and energy dissipation. It is also demonstrate that, under moderate and high magnitude earthquakes, the damage is concentrated in the added dampers and the response of the existing RC framed structure is chiefly elastic.

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