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RETROFITTING PRECAST FRAME STRUCTURES UNDER EARTHQUAKE LOAD BY MONOLATERAL DISSIPATIVE BRACES

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Abstract

Seismic retrofitting of existing precast concrete frame structures built according to null or inadequate seismic design provisions is a main challenge for the Engineering Community. The typical large spans and heights of these buildings do not allow for the installation of current bilateral dissipative braces, including buckling-restrained braces, due to the unviable brace length required.

To this aim, an innovative seismic retrofitting technique based on mono-lateral dissipative bracing is employed. These devices, introduced in the recent years in fields of Engineering other than Earthquake, dissipate energy through friction or plasticisation in tension, while freely deforming in compression. This technique is studied for the seismic retrofitting of a typical precast concrete industrial frame structure representative of the European heritage, among others.

Non-linear dynamic time history analysis is employed in this work for the evaluation of the seismic performance of the original building structure and the one retrofitted with highly nonlinear monoliteral braces with different threshold activation loads. The results show the potential benefits of this promising retrofitting technique in providing a complete protection from the earthquake preventing the structure from attaining yield and damage even when subjected to strong motions.

Keywords: Seismic risk, Precast structures, Probabilistic assessment, Panel connection systems

1 Introduction

Precast concrete structures are diffused all over the world due to their beneficial features, like fast construction, ease of erection, high-strength materials, quality control, long span provisions and many more. In Europe as well as in other continents, long-span precast frame roof structures have been extensively employed to cover industrial activities. As a matter of fact, many precast industrial buildings were built in the past either neglecting any seismic design provision, or designed according to obsolete seismic criteria, and/or designed according to seismic actions than those proposed in the current regulations. This highlights the need for systematic seismic retrofitting of a large portion of the precast built heritage in many buildings, which turns being a challenge for both the technical and the civil communities. This need was also dramatically pointed out by the poor structural performance of precast buildings during the Emilia earthquakes occurred in Northern Italy in 2012, as well as by the poor performance of the cladding panel system observed after both the former and the L'Aquila earthquake occurred in Central Italy in 2009 [1-6].

To accomplish this challenge, growing research efforts were being spent in proposing and studying retrofitting solutions for this type of buildings [7-15]. Among the envisaged solutions, particularly promising would be the post-installation of dissipative braces, which can add relevant stiffness and act as fuses incorporating large amounts of energy dissipation to the typically flexible structural behaviour of these frames, reducing their drift and subsequently their damage under seismic loads. However, current technologies employing dissipative braces are mostly based on bilateral dissipative devices which dissipate energy in both tension and compression according to different mechanisms (purely viscous, visco-elastic, hysteretic, etc.), which require buckling of the brace to be avoided in compression, leading to bulk brace profiles and to high costs, in addition to absolute limitations in length of about 6-8 m. Especially the last feature of these devices makes their application for the retrofit of precast structures practically difficult, due to the long spans and heights typically associated with these buildings, where standard systems are characterised by maximum span length of up to 50 m for beams and 42 m for roof elements [16].

Proposals to employ monolateral dissipative devices were recently introduced by [17] to mitigate blast actions in steel structures and by [18] to mitigate seismic actions in steel structures. Both proposed devices are based on a mechanical ratcheting mechanism which allows a mechanical interlock to restrain the displacement in tension of a steel bar which can yield and dissipate energy by plastic hysteresis, and allows for relative sliding with negligible actions in compression thanks to the ratchets. Moreover, very recently a device whose dissipation mechanism is based on friction was proposed by [19].

Monolateral dissipative devices can be installed in relatively slender braces in order to be employed for the retrofitting of long-span precast concrete industrial frame structures. The present paper investigates the potential benefits of their installation in existing structural assemblies by analysing the seismic performance of a case study typical precast industrial building in its original and braced configurations. Different threshold loads associated to either yielding or slippage for plastic and friction devices, respectively, are addressed. Due to the high mechanical non-linearity of the brace (and frame at large drift) behaviour and to the strong influence of geometric non-linearity of the frame caused by its relevant flexibility, non-linear analysis was carried out on a plane frame finite element modelling of the structure, encompassing both static and dynamic time history analyses.

2 Monolateral dissipative braces

Post-inserting dissipative bracings in the frame structure of a precast frame building significantly alters its original lateral load resisting system by locally introducing remarkable stiffness and fuse energy dissipation, hence this sort of intervention shall be carefully planned and designed. Among the different possible arrangements of bracing elements, some solutions are identified in Figure 1.

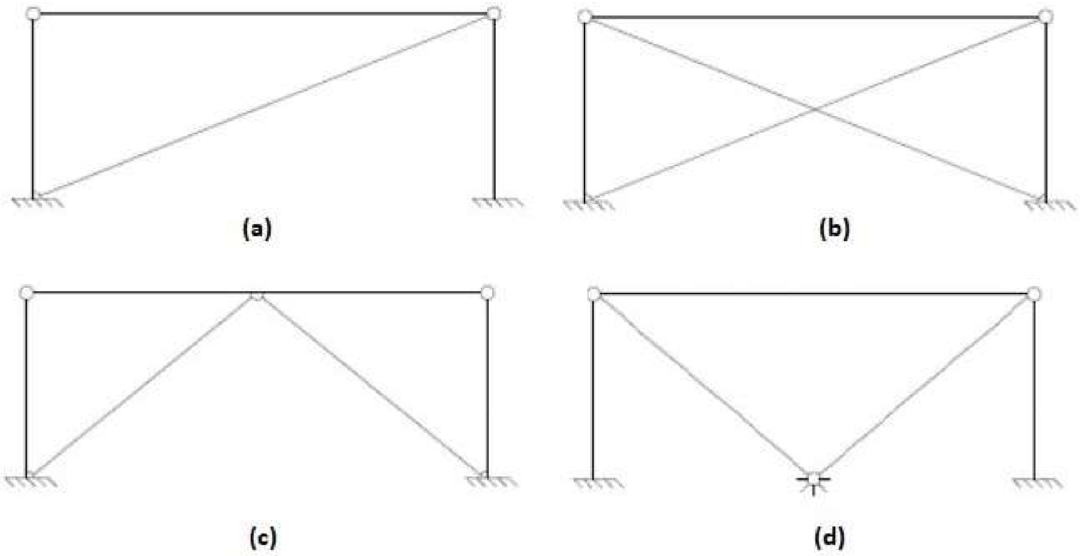


Figure 1 Typical bracing configurations: (a) single diagonal, (b) cross, (c) chevron, (d) V-shaped

The configurations identified as single diagonal, cross or V are cluttering the span of the portal, while the only one among the basic ones identified in Figure 1 allowing for only partial span occupancy is the Chevron. However, the latter has the disadvantage of not being concentric, that is to solicit the beam in its center line. Considering the application of monolateral braces to existing structures, this configuration is however not recommended, due to the interaction of the kinematics of braces and beam when subjected to gravity loading, which could jeopardise its functionality.

The kinematics of the different typologies of bracing previously identified when subjected to lateral drift is sketched in Figure 2 and the behaviours at different stages of a cyclically imposed remarkable lateral displacement are described in Table 1. When referring to standard monolateral non-dissipative braces, the deviation from the ideal behaviour to the more realistic is due to the actual behaviour of the brace in compression, which is characterised by some stiffness and hysteresis, although predominantly influenced by buckling, as observed in many experimental activities, including [20,21].

The device considered in this research is unidirectional, i.e. active only in tension. Physical devices following this concept were proposed in literature by [17,18]. Both devices are based on the use of dissipative braces for the plasticization of the steel with a rack mechanism which allows free shortening in compression and steel plasticisation in tension after effective locking (Figure 3).

As previously mentioned, also devices based on friction have been proposed [19]. The current work considers an ideal system as described in Figure 2c, based on a perfect elastic-plastic behaviour which could be attributed to either plasticisation or friction mechanisms. Moreover, different values of the yielding/friction threshold loads are considered, ranging from 50 kN to 200 kN. Since no direct reference is made to existing devices in the technical and scientific literature, this work is aimed at investigating possible future configurations of dissipative devices, identifying the potential impact of different threshold loads over the seismic performance of the retrofitted building and fostering their evolution and diffusion.

Table 1 Kinematics of different typologies of braces under imposed cyclic lateral displacement. Energy dissipation by plasticity of steel is assumed.

Step	Standard bilateral braces		Dissipative bilateral brace	Dissipative monolateral braces	
	Right brace	Left brace		Right brace	Left brace
Rightward displacement	Slight buckling	Yielding and plastic elongation	Yielding and plastic elongation	Shortening	Yielding and plastic elongation
Back to origin	Regain of initial shape	Slight buckling	Yielding and plastic shortening	Yielding and plastic elongation	Shortening
Leftward displacement	Yielding and plastic elongation	Strong buckling	Yielding and plastic shortening	Yielding and plastic elongation	Shortening
Back to origin	Slight buckling	Slight buckling	Yielding and plastic elongation	Shortening	Yielding and plastic elongation
Next cycles	Negligible energy dissipation		Robust hysteresis and strong energy dissipation	Robust hysteresis and strong energy dissipation	

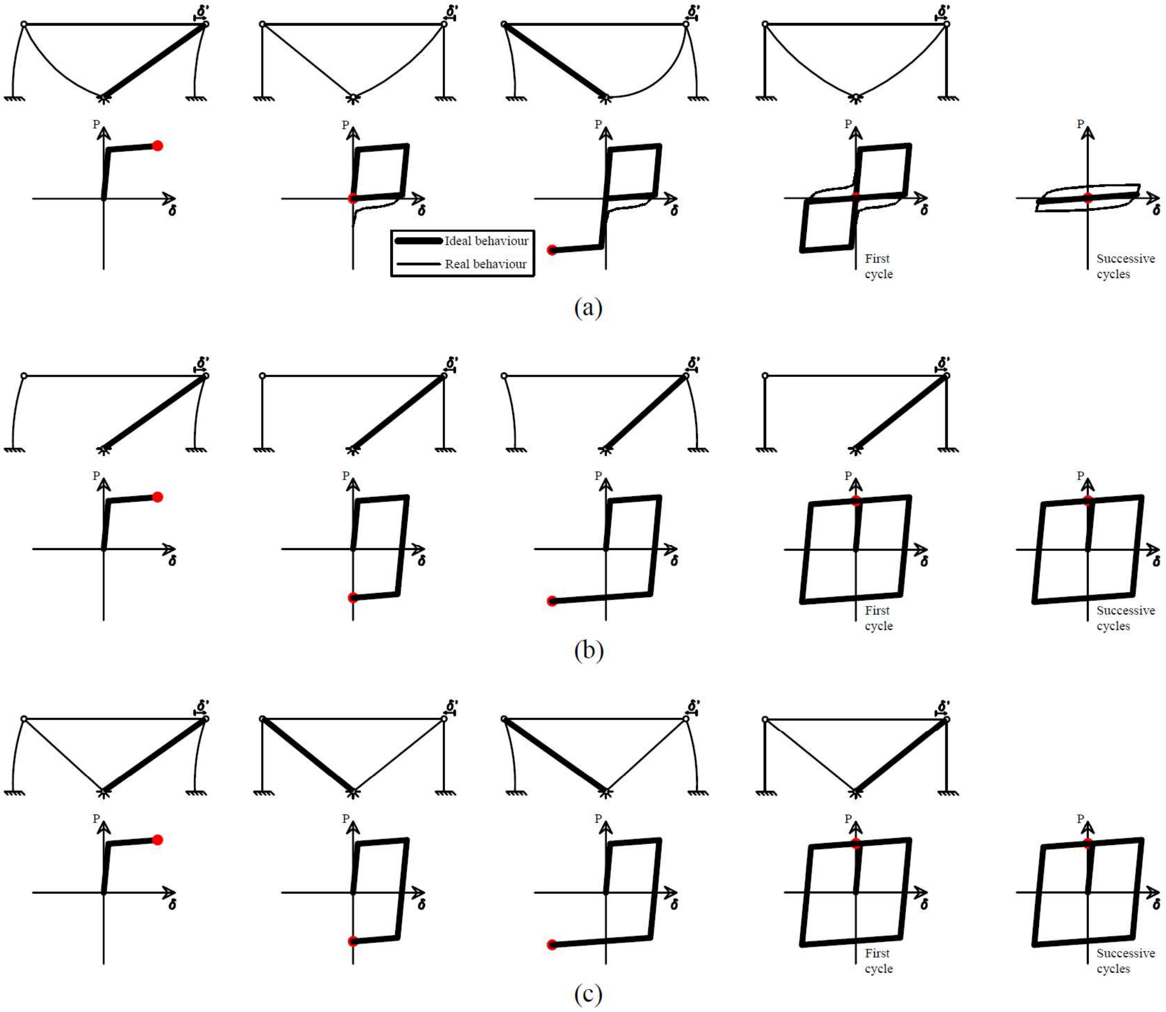


Figure 2 Elasto-plastic hysteretic behaviour of (a) traditional slender coupled braces acting in tension only, (b) bilateral dissipative single brace acting in both tension and compression, (c) dissipative coupled braces acting in tension only

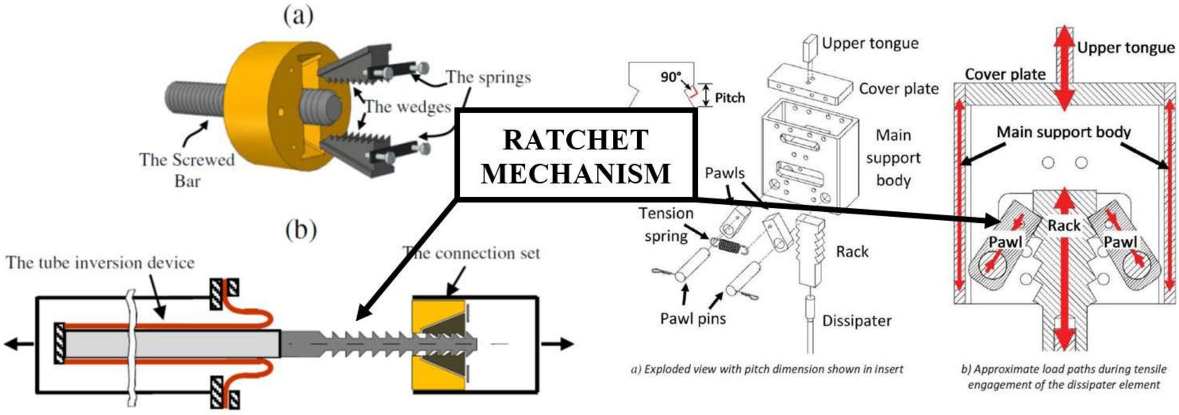


Figure 3 Hysteretic monolateral dissipative devices based on ratcheting mechanism: UPD (Monir 2013 – left) and GnG1 (Cook et al. 2018 – right)

3 Case study industrial building

To study the effectiveness of the proposed dissipative braces in precast industrial buildings, a typical single storey multi-bay precast frame structure was considered as a case study. The case study structure is made with a typical precast system for industrial construction [16] made by rectangular/square columns, H-shaped prestressed beams, wing-shaped prestressed roof elements, and completing barrel/shed vaults. Front and side views of the building are shown in Figure 4, and synthetic data of the building are listed in Table 2. Figure 4 also shows a proposal for the retrofitting intervention, consisting in the insertion of a couple of dissipative braces equipped with monolateral devices per frame in both transversal and longitudinal direction of the building. This quantity of braces is deemed necessary due to the assumed lack of diaphragm rigidity of the roof deck [22,23], which could jeopardise their benefits, although a deeper study considering the whole structural assembly and the actual deformability of the deck could lead to the adoption of less braces and to an optimisation of their distribution.

The column reinforcement details are collected in Figure 5.

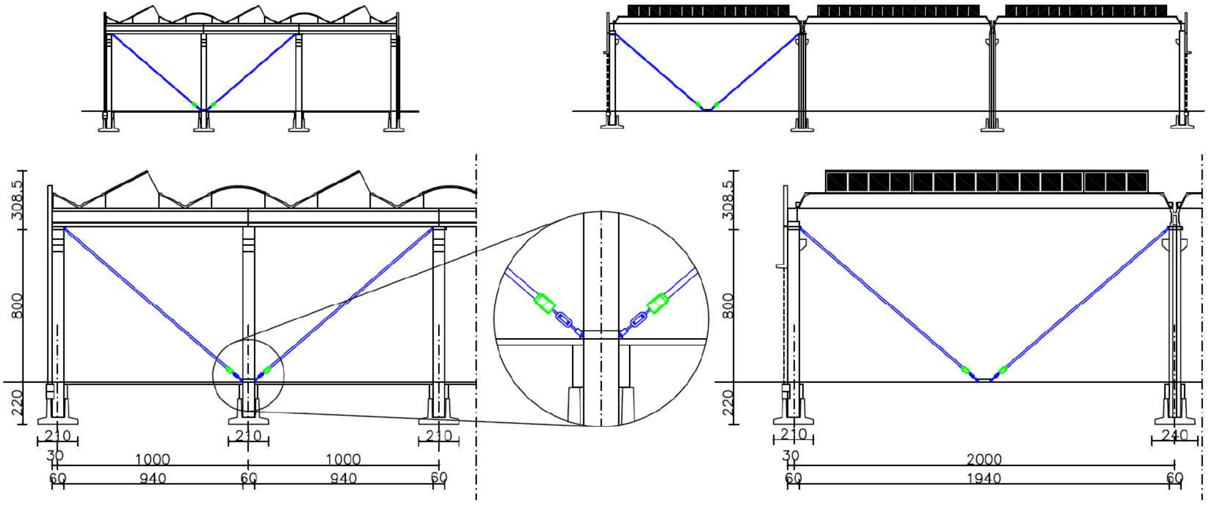


Figure 4 Structural side views of the case study building

Table 2 Synthetic data of the case study building

Column	side length	0.6 m
	clear height	8 m
	longitudinal steel ratio	1 %
Frame layout	grid size	20 m x 10 m
	spread mass	300 kg/m^2
	no. of bays / naves	3 / 3
	diaphragm	flexible
	cladding panel arrangement	isostatic
Material	concrete strength f_{ck}	45 MPa
	reinforcing steel yield strength f_{yk}	450 MPa
	brace steel yield strength f_{yk}	235 MPa
Dissipative device	initial stiffness	rigid
	threshold load	Variable 50~200 kN
	brace length	12.8 m

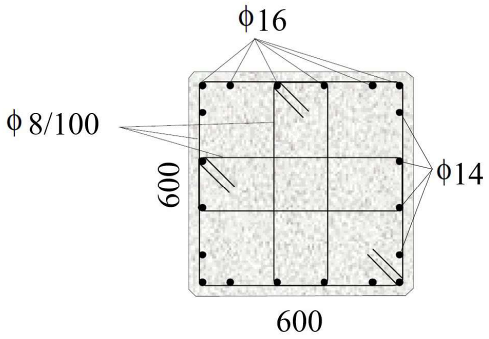


Figure 5 Column cross-section (dimensions in mm)

4 Non-linear analysis

To study the effectiveness of proposed dissipative devices in precast structures, a set of non-linear analyses has been carried out on the basis of finite element models developed with the software Straus7 [24]. The models were referred to one plane frame of the case study building, in the direction of the beams. However, the results are also representative of the frame oriented in the direction of the roof elements, since the braces have the same inclination, the number of columns is the same, the mass is the same, and the columns have symmetrical cross-section with only limited differences in terms of reinforcement layout in the two main directions. Two models were set, as shown in Figure 6.

The model of the original frame was made by beam elements fully fixed at their bases (10 elements per column having equal length of 80 cm), to which non-linear moment-curvature diagrams were attributed. Equal displacement was imposed along the different columns by coupling links. Lumped masses were attributed to the top node of the beam elements simulating the columns, and the value of mass was evaluated on the basis of the assumed geometry of the building considering the structural weight only. The mass from additional non-structural weights, such as thermal insulation and waterproofing layers, was neglected due to its small entity. The same mass was attributed to the central and the edge columns, where the halved structural weight from the roof is compensated by the tributary mass of the suspended horizontal sandwich cladding panels made of precast concrete. Simplified three-linear moment-curvature relationships were attributed. The first branch was related to the plain sectional stiffness up to the cracking moment; the second branch was related to the cracked stiffness evaluated as the line linking the cracking and the yielding points, where the latter was evaluated on the basis of the non-linear sectional equilibrium after having attributed a Sargin-Saenz stress-strain relationship to concrete and an elastic-plateau stress-strain relationship for steel; the third indefinite horizontal plateau branch simulated the post-yielding sectional behaviour. Nominal strength properties were considered. Takeda hysteretic rule was attributed to the elements.

The model of the retrofitted frame was implemented by adding the braces. They were simulated by placing in series two elements: a non-linear axial spring and a tension-only contact element. A perfect elastic-plastic force-displacement relationship was attributed with the considered threshold value and an elastic stiffness equal to 30 kN/mm simulating the elastic axial stiffness of a round tubular steel profile connecting the dissipative device to the top and bottom of the adjacent columns. A kinematic hardening hysteresis was attributed.

The results from non-linear static analyses are shown in Figure 7 with reference to both the curves neglecting and considering the effects of 2nd order P-δ effects. It can be recognised that they play a remarkable role especially over the softening descending branch. It is however reminded that many columns of existing buildings are not provided with the transverse reinforcement recommended by current regulations, and that a lack of confinement could bring to early failure by rebar buckling under cyclic loading, which means that hysteresis in the third plastic branch may not be stable and lead to failure before the typical flexural failure at the basis of properly confined precast columns occurring when the 2nd order moment overwhelms the resisting one.

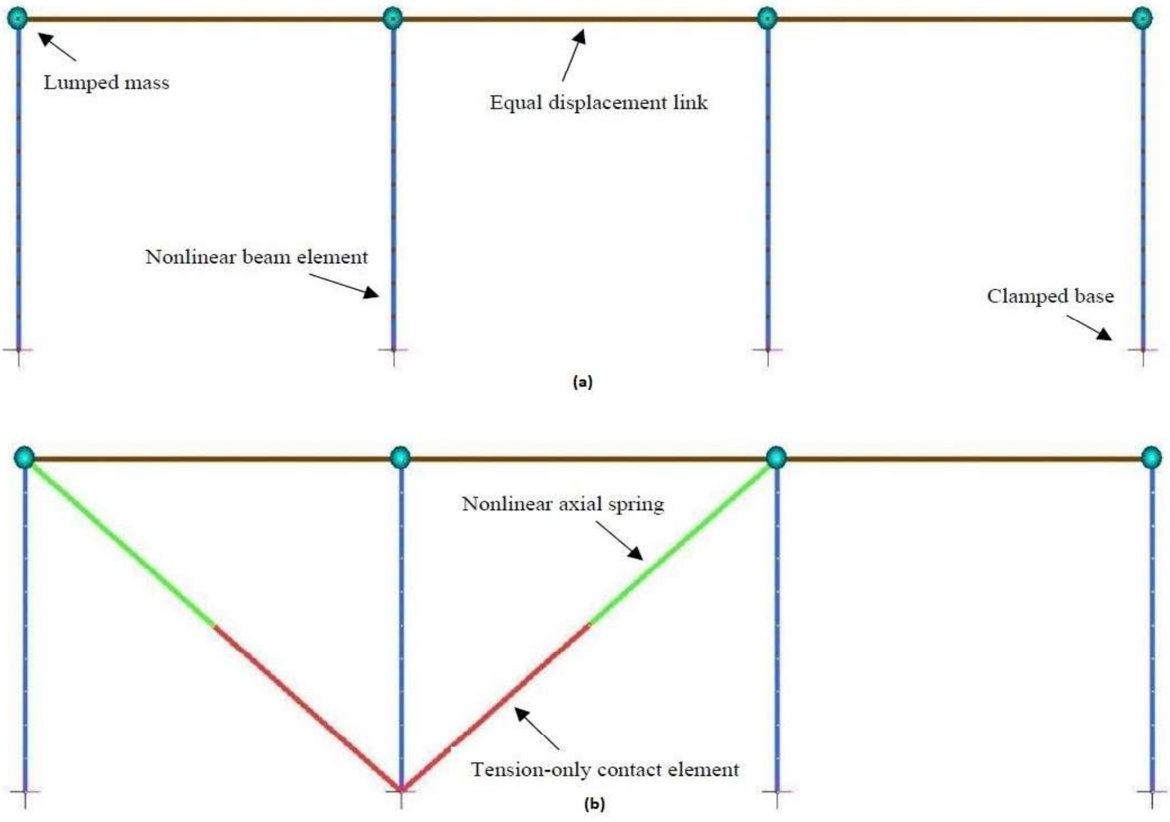


Figure 6 Structural finite element models: (a) original unbraced frame, and (b) retrofitted braced frame.

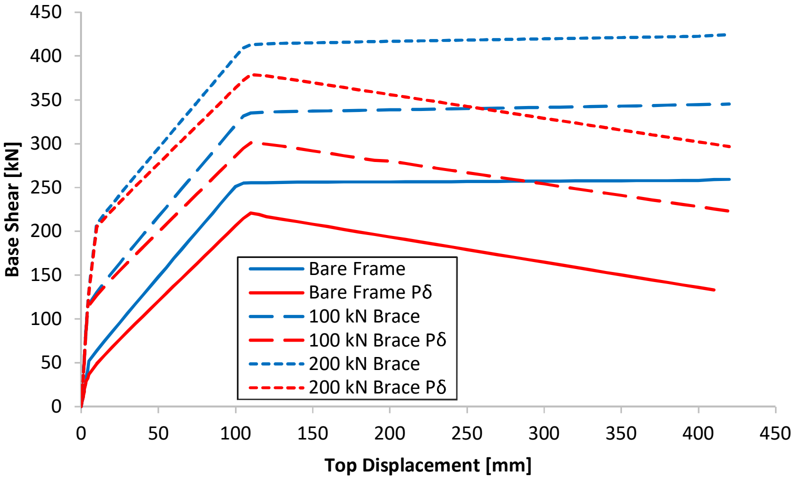


Figure 7 Results from non-linear static pushover analysis

A set of 10 different artificially generated accelerograms compatible with the elastic spectrum (type 1, subsoil class B) of Eurocode 8 [26] (Figure 8) have been employed to carry out non-linear dynamic time history analyses. Rayleigh damping of 2% was attributed to a wide range of frequencies.

The main results are plotted in Figure 9 in terms of maximum top displacement and maximum base shear attained during the non-linear dynamic time history analyses. It is observed that the bare frame structure attains yielding for all accelerograms with PGA equal or higher than 0.2g, whereas the braced structure with device capacity of 100 kN attained yielding for only 20% of the total accelerograms scaled at PGA of 0.4g and 100% for PGA of 0.5g, and the braced structure with device capacity of 200 kN attained yielding for only 10% of the total accelerograms scaled at PGA of 0.5g.

The activation of the devices, corresponding to a top structural displacement of about 4 mm and 8 mm for 100 kN and 200 kN of threshold loads, respectively, are overwhelmed for all the considered analyses with retrofitting bracing, meaning that the devices always entered into function and provided dissipation of energy by hysteretic damping.

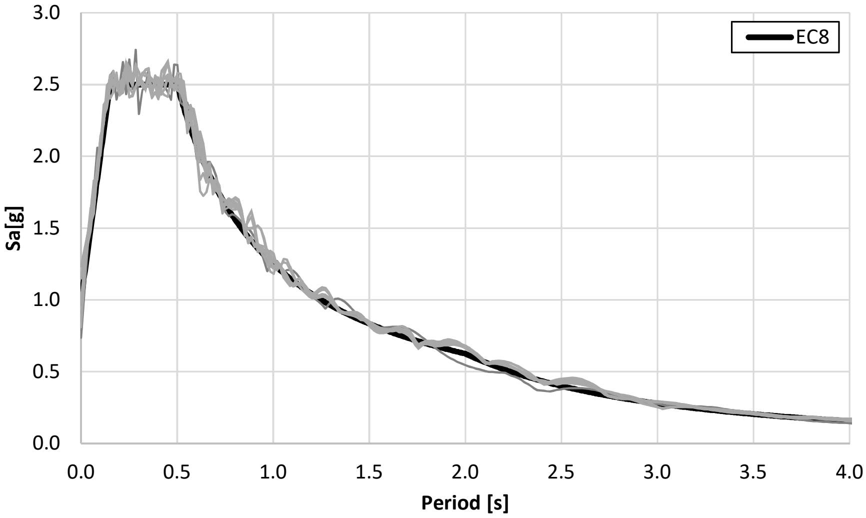
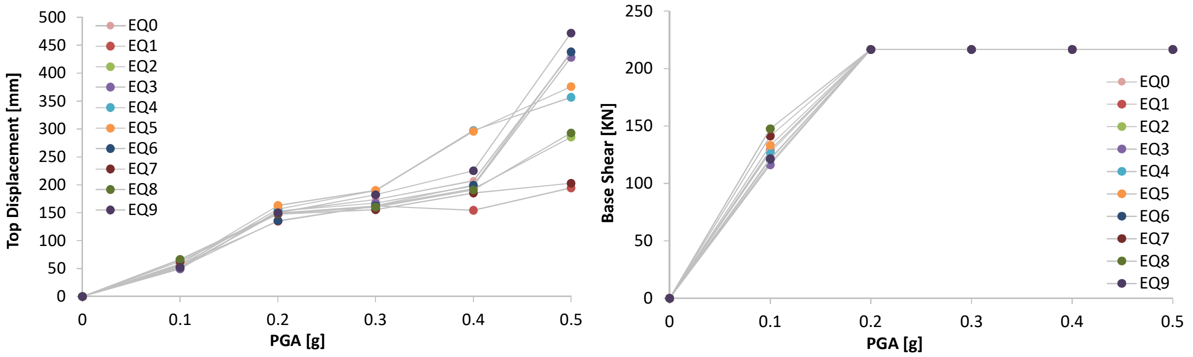
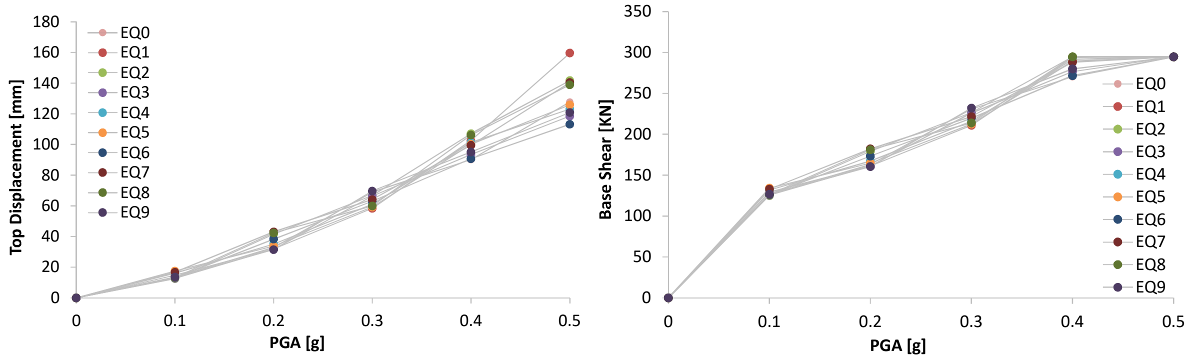


Figure 8 Response spectra of the signals adopted compared with the Eurocode 8 elastic spectrum for subsoil type B (plotted with unitary soil factor S and damping correction factor η).

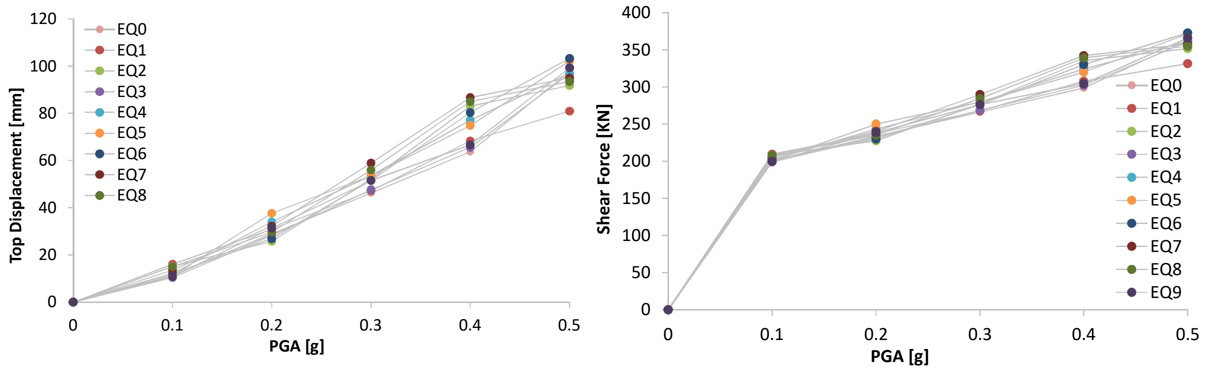
Bare frame structure



Braced frame with device threshold load 100 kN



Braced frame with device threshold load 200 kN



Comparison of mean values

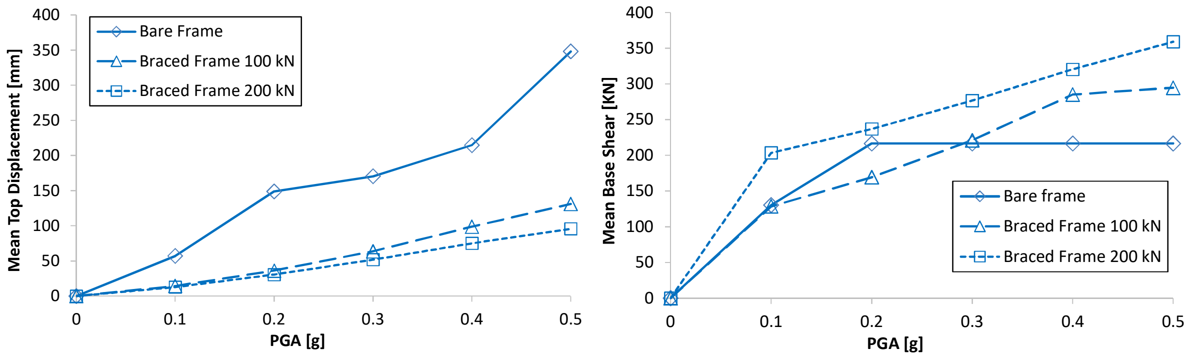


Figure 9 Results of non-linear dynamic time history analysis

The comparison of the mean maximum base shear attained during the analyses shows the interesting phenomenon of equivalence for bare frame and retrofitted structure with device capacity of 100 kN at levels of PGA of 0.1g and 0.3g, despite dramatic reductions of mean maximum displacement of 65% and 47%, respectively. Comparing the same configurations, even a reduction of the maximum base shear of 22% was found for PGA of 0.2g, associated to another dramatic drop of the displacement of 52%. For higher PGAs and for higher device threshold load, the mean values of the maximum base shear attained by the braced structure are higher than the one attained by the bare frame structure, although it is reminded that shear in the bare frame structure is associated to a strong bending occurring at the base of the columns, whilst for the retrofitted case it is predominantly transmitted as localized shear action in correspondence of the base of the brace. Moreover, the shear transmitted to the base of the devices, whether it is transmitted via a connection to the industrial concrete pavement (braces in the direction of the roof elements) or by the column base (braces in the direction of the beams), is thresholded by the activation of plasticity or slippage of the devices to values easily compatible with the strength of industrial pavements or with the increase of shear actions in the precast column elements.

It is also reminded that the hysteresis of the columns in plastic branch can be unstable, as previously mentioned, and therefore on the safe side ultimate displacements can be determined as associated to the yielding top displacement (about 10 cm) for poorly confined columns, which further highlights the benefits introduced by the proposed retrofitting technique, where structures failing for earthquakes with PGA in between 0.1g and 0.2g would be fully protected up to catastrophic earthquakes with PGA up to 0.5g.

5 Conclusions

A recently developed technology for the construction of monolateral dissipative braces in the steel construction industry was proposed to be employed for the seismic retrofit of existing precast concrete industrial frame structures suffering from design deficiencies. The case study analysed by considering a typical precast structure with span and height lengths not compatible with the installation of other types of dissipative braces showed a very high potential efficacy in mitigating the seismic drifts while thresholding the base shear. The envisaged solution could represent a viable retrofitting technique to farther the risk of collapse of the structural arrangement, often poorly detailed at the column bases and not relying on a stable hysteresis in plastic field. Moreover, the issues related to the interaction with non-structural elements, such as cladding panels, which currently represents a primary concern for those structures, would also be dramatically mitigated by the reduction of structural drift. Similarly to all structural applications which strongly modify the lateral load resisting system of any structural assembly, the proposed retrofit intervention shall be accurately designed for real applications. Non-linear dynamic time history analysis is suggested as the method to be employed for this task, also due to the strong mechanical non-linearity of the dissipative device, either it is based upon plasticisation or friction. Potential issues for the application of this technique, in addition to the encumbrance of the braces, may be related to structures not relying upon a rigid diaphragm action, which is indeed very common for the structural typology at study. Moreover, the roof connections (beam-to-column and roof-to-beam) shall also be adequately checked or strengthened considering the concentration of shear load in correspondence of the braces, although thresholded by the dissipation mechanism.

Future research activities will be devoted to the physical development and to the mechanical characterisation of the device based upon friction, and on the development of simplified design methods which, although not fully replacing non-linear dynamic time history analysis, could however relegate the employment of the latter for check matters, rather than for full design processes.

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