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Performance evaluation of a seismic retrofitted R.C. precast industrial building

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Abstract. Recent seismic events occurred in Italy (Emilia-Romagna 2012, Abruzzo 2009) and worldwide (New Zealand 2010 and 2011) highlighted some of the weaknesses of precast concrete industrial buildings, especially those related to the connecting systems traditionally employed to fasten the cladding panels to the internal framing. In fact, one of the most commons fails it is possible to observe in such structural typologies is related to the out-of-plane collapse of the external walls due to the unsatisfactory behaviour of the connectors used to join the panels to the perimeter beams. In this work, the strengthening of a traditional industrial building, assumed as a case study, made by precast reinforced concrete is proposed by the adoption of a dual system allowing the reinforcement of the structure by acting both internally; by pendular columns and, externally, on the walls. In particular, traditional connections at the top of the walls are substituted by devices able to work as a slider with vertical axis while, the bottom of the walls is equipped with two or more hysteretic dampers working on the uplift of the cladding panels occurring under seismic actions. By means of this approach, the structure is stiffened; obtaining a reduction of the lateral drifts under serviceability limit states. In addition, its seismic behaviour is improved due to the additional source of energy dissipation represented by the dampers located at the base of the walls. The effectiveness of the suggested retrofitting approach has been checked by comparing the performance of the retrofitted structure with those of the structure unreinforced by means of both pushover and Incremental Dynamic Analyses (IDA) in terms of behaviour factor, assumed as a measure of the ductility capacity of the structure.

Keywords: precast concrete structures; reinforced concrete; industrial buildings; dampers; seismic analysis

1. Introduction

In last decades, a large amount of buildings with prefabricated structure has been realized worldwide. As in other countries, in Italy, prefabricated technologies have been applied mainly for the construction of industrial facilities (the 80% of the Italian industrial buildings heritage is realized with prefabricated structures) and in minor part for the realization of commercial and residential multi-storey buildings (RELUIS, 2012). The recent seismic events occurred in Italy, (Abruzzo 2009, Emilia-Romagna 2012) (De Iuliis *et al.* 2010, Magliulo *et al.* 2014) have led to the collapse of a high number of industrial buildings, bringing the attention on the safety of precast concrete structures against seismic events (Belleri *et al.* 2015a). In addition, other recent earthquakes occurred in New Zealand (2010 Mw=7.0 and 2011 Mw=6.1) and also in Italy (20th and 29th of Maj 2012, Mw=5.9 and L'Aquila quake) highlighted that the main collapse modes experienced by precast reinforced concrete residential and industrial building flexible structures, whose behavior is generally governed by limiting the p-delta effects and controlling the displacement demand, were related to failures in connections at the base, giving rise to soft storey mechanism, as a main consequence of the first quake and second aftershock (Belleri *et al.* 2015b, Liberatore *et al.* 2013, Smyrou *et al.* 2011, Babič and Dolšek 2016). One of the most significant reasons leading to the collapse of structures is attributable to the unsatisfactory behavior exhibited by the joints traditionally adopted to fasten the top of the cladding panels to the perimeter beams of the internal framing. In fact, during these earthquakes is usual to observe that facades of industrial structures fail out-of-plane due to the tearing of the top connections, which are the only constraints usually adopted to fix the cladding panels to the internal framing. Such connections are ideally conceived to uncouple the in-plane motion of the cladding panels from that of the internal structure and to fix the out-of-plane overturning of the walls. In fact, they are made by two sliding guides where the first one is located on the top of the perimeter beams and the second one on the inner face of the external walls. In addition, particular types of hammerhead bolts placed inside the channels absorb the out-of-plane actions (Fig. 1). Even though these systems have been widely used, after the most recent earthquakes, an unforeseen behavior has been observed. In fact, the eccentricity existing between cladding panels and perimeter beams of the internal framing has led, during the seismic motion, to the achievement of kinematic mechanisms not compatible with the movements allowed by the connectors. Consequently, the typical failure modes exhibited by such connections have been essentially characterized by the fracture of the bolt shank or by the

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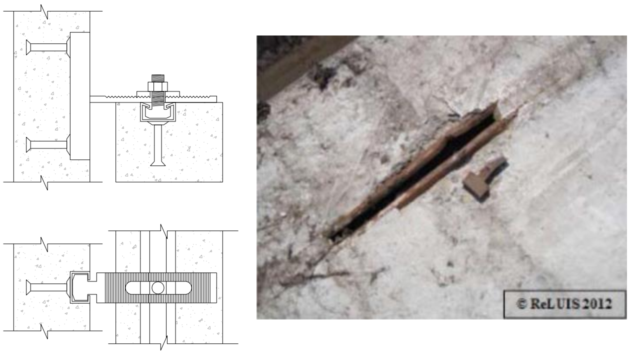


Fig. 1 Classical Connector for Cladding Panels of Industrial Buildings, scheme and photo

tearing of the edges of the steel channels, due to the locking of the bolt heads within the horizontal and vertical guides. In addition, contrary to the expected, the locking of connections has produced also the collaboration between cladding panels and internal structure leading to a reduction of the natural period of vibration with a consequent increase of the entity of seismic actions the structure has to face during an earthquake. Within this framework, in this paper, a solution able, on one hand, to overcome the problems related to the out-of-plane failure of the walls, and from the other hand, to exploit the external cladding panels, traditionally considered as non-structural elements, is proposed.

This coupling is obtained by substituting the traditional joints at the top of the walls with a connection able to constrain the horizontal in-plane and out-of-plane degrees of freedom of the panels and to work as a sliding guide in the vertical direction in order to allow the rocking motion (Henry and Roll 1986, Mangiulo *et al.* 2015, Zoubek *et al.* 2016). In addition, two or more hysteretic dampers already tested in a previous activity (Latour and Rizzano 2012), namely XL-Stubs, are located at the bottom of the panels, working on the uplift of the cladding panels occurring under seismic actions. Thanks to the participation of the cladding panels, the structure is stiffened, obtaining a reduction of the lateral drifts under serviceability limit states and its dissipative capacity is improved due to the additional source of energy dissipation represented by dampers located at the base of the walls (Castaldo and Tubaldi 2015, Castaldo *et al.* 2015, Latour *et al.* 2015, Silvestri *et al.* 2011, Belleri *et al.* 2016, Bora *et al.* 2007, Scotta *et al.* 2015).

The obtained structure has a big dissipative capacity being the resulting behaviour factors comparable with the ones obtained by other design procedure able to fully exploit dissipative capacity of structures (Montuori *et al.* 2014, Formisano *et al.* 2015, Piluso *et al.* 2014, Montuori *et al.* 2015, Montuori and Muscati 2015). The reliability of the suggested approach has been verified by means of both push-over and dynamic analysis on a case study building retrofitted by using the proposed techniques.

2. Proposed dual system

As discussed previously, prefabricated concrete buildings are designed by neglecting the influence of the cladding panels. In fact, such elements are normally considered secondary elements with no influence on the lateral stiffness and contributing to the structural response only in terms of mass. However, recent studies have revealed that the influence of the cladding panels on the seismic response of prefabricated buildings cannot be neglected (Scotta *et al.* 2015). In fact, the external walls can significantly affect the seismic response of the building by changing natural vibration period, member forces distribution and ductility supply. After recent failures, the problems affecting the cladding panels connections has been faced following two different approaches: the first one has the scope to provide an out-of-plane "emergency" constraint for the panels, while, the second one is aimed at the substitution of the existing connecting elements with new ones able to accommodate the vertical and horizontal displacements arising under seismic loads.

However, these approaches are still unsatisfactory and do not take in account the panel contribution. Furthermore, even though the aforementioned approaches partially solve the problems related to the premature failure of the connections, they are not able to solve the problem of the uncoupling between the internal structure and the cladding panels. This issue is of paramount importance because it affects the period of vibration of the building and consequently the seismic actions arising in the structural elements. Therefore, a complete modelling of the structure, comprehensive also of the cladding panels, should be carried out in order to accurately estimate the performance of the building under seismic actions.

Within this framework, the solution proposed (Latour and Rizzano 2012, Latour *et al.* 2015) is an alternative to the classical design philosophy. It consists in the substitution of the existing connectors of the cladding panels with new ones able to realize a dual system,

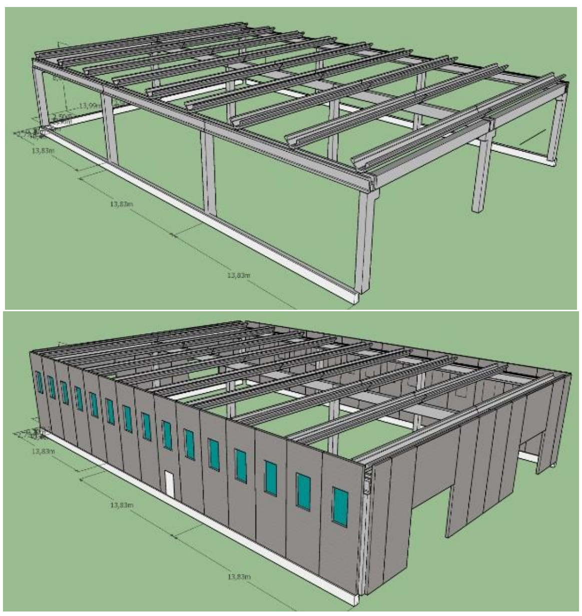


Fig. 2 Traditional Industrial Building. Top: Bare Frame, Bottom: Frame with cladding panels

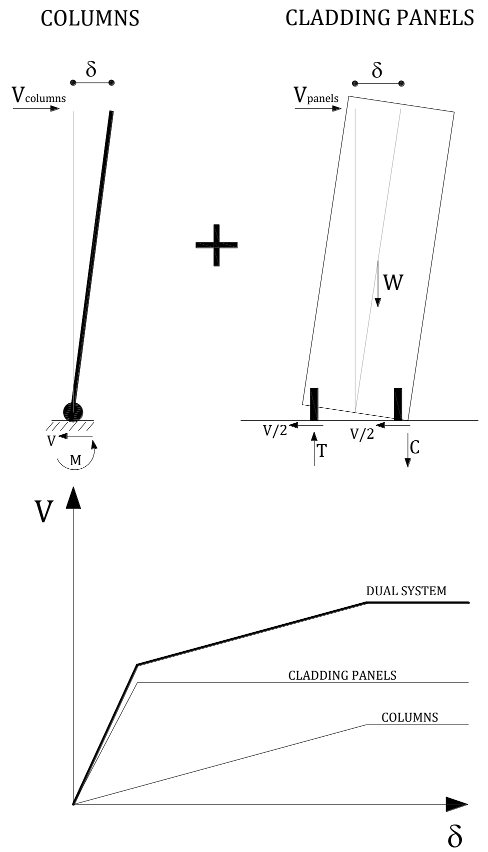


Fig. 3 Idealization of the dual system composed by columns and cladding panels

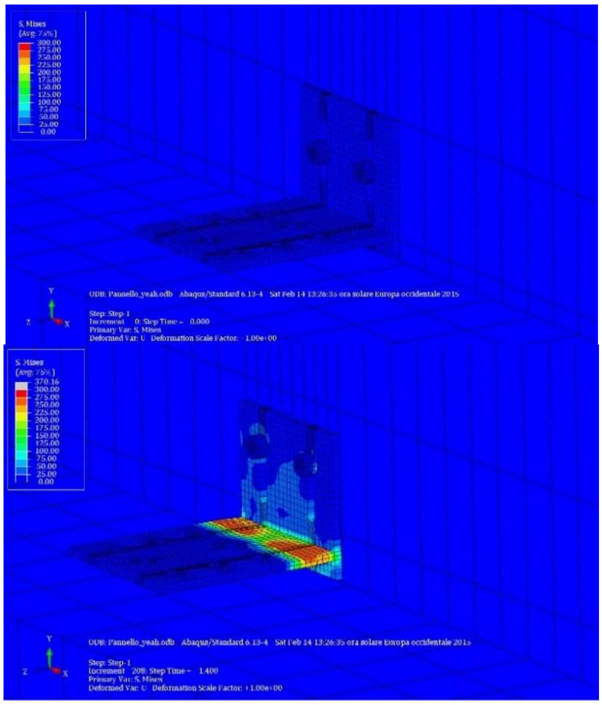


Fig. 4 Vertical movement of the top connection under seismic actions. Preliminary model

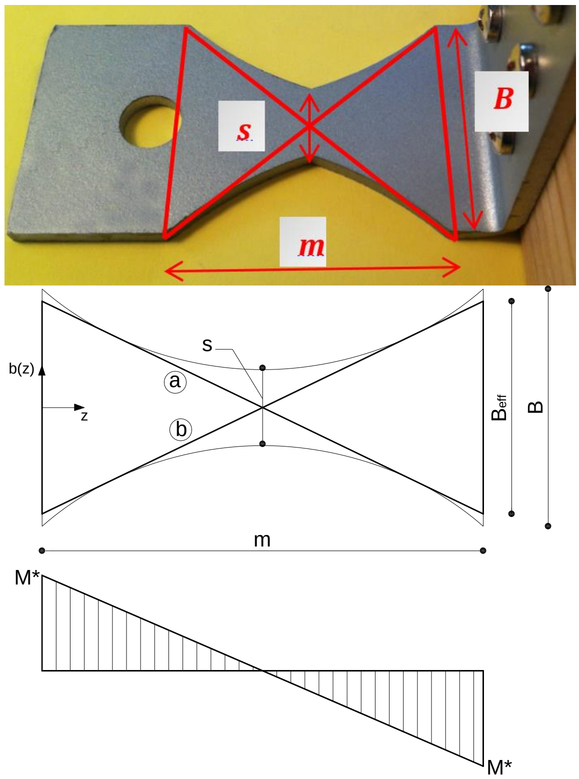


Fig. 5 Bending Moment Diagram arising on the tapered flange of the patented damper

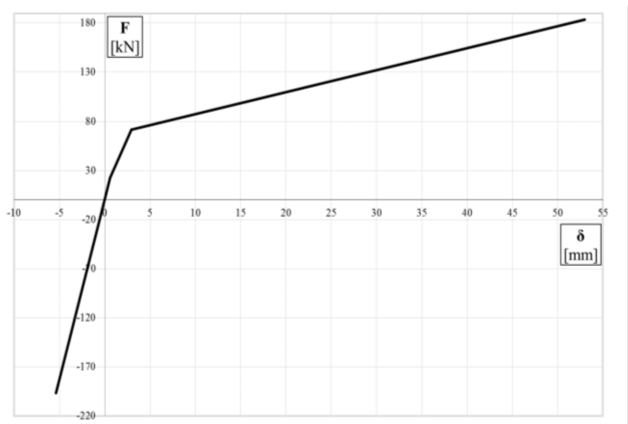
composed by the external walls and the internal cantilever columns connected by pendula, working in parallel. In order to realize such a hybrid system, authors propose to connect the cladding panels to the perimeter beams and to the foundations by means of two device typologies located at the base and at the top of the walls. In particular, the top of the panels are connected to the internal structure by means of a device able to restrain the out-of-plane direction and the in-plane horizontal translation and to leave free the vertical in-plane direction in order to allow the rocking motion of the cladding walls.

These top-connections (Fig. 4) allow the rocking motion of the walls by fastening the cladding panels to the perimeter beams of the internal framing have to be designed in order to restrain the out-of-plane and the in-plane horizontal direction leaving free the vertical translation. In this way, under seismic actions, the shear load can be transferred from the internal framing to the external walls. In fact, provided that the cladding panels are designed in order to be relatively stiffer than the internal framing, almost all the seismic action is resisted by the external walls, which are connected to the foundations of the building by means of the hysteretic dampers located at the bottom of the walls (XL-stubs). These last ones are able to absorb shear and axial forces are located at the base of the panels (Fig. 3). XL-stubs have to be designed to oppose the actions transferred by the connections located at the top of the walls and to dissipate the seismic energy by means of hysteresis loops activated by the alternate rocking motion of the cladding panels during a seismic event (Brunesi *et al.* 2015).

These dampers (Fig. 5), that have been recently patented (Latour *et al.* 2013) are conceived as metallic hysteretic devices working in double curvature, such as ADAS devices (Added Stiffness And Damping). The main idea is to concentrate the dissipative zone of the Hold-down in the flange plate. To this scope, the stem zone is over-strengthened by adopting a proper number of connectors in order to concentrate yielding in the flange plate. In addition, the flange plate of the angle is weakened in order to obtain an hourglass shape very similar to that usually adopted in ADAS devices (Latour 2016). In fact, it is easy to understand that if the flange plate of the angle is cut providing a law of the width which varies accordingly with the diagram of the bending moment it is ideally possible to get the contemporary plasticization of all plate sections obtaining a ductility demand distributed along the whole plate. Starting from the diagram of the bending moment arising in a plate, under the assumption of a strong bolt and a weak plate, it is easy to verify that the shape which provides the simultaneous plasticization of all plate sections is an ideal X-shape (Fig. 5).

In the past experimental activity, the XL-stub has already been tested under monotonic and cyclic loading conditions in order to evaluate its stiffness, resistance and hysteretic behavior (Latour and Rizzano 2015a, b). In addition, by carrying out constant amplitude cyclic tests the fatigue life curve of the device has been evaluated.

In the past experimental program, the mail mechanical parameters have been determined and, in the particular, the plastic resistance has been obtained according to the procedure proposed by EN 1993-1-8, i.e., in correspondence of the intersection of the experimental curve with a line of slope equal to one third of the initial stiffness. The monotonic envelope and the force-displacement parameters of the XL-stub are reported in Fig. 6.



δ mm	F kN
-4.09	-150.00
0.00	0.00
0.62	22.58
2.94	71.46
53.00	183.41

Fig. 6 XL-Stub monotonic envelope and Force-displacement parameters

Under significant seismic events the XL-stub located at the base of the external walls supply to the energy dissipation at low displacements, protecting the columns from damages (Fig. 3). In this way, if the cladding panels are designed to be sufficiently stiff with respect to the internal columns, the seismic action is entrusted to the external walls, while the internal columns have to support the vertical loads only and follow the lateral displacements remaining essentially in elastic range. Anyway, if the column behaviour is still unsatisfactory many techniques should be adopted for the retrofitting of the single member (Montuori and Piluso 2009, Montuori *et al*. 2012, Montuori *et al*. 2013).

3. Case study

3.1 Structure without reinforcements

In order to evaluate the seismic performance of the proposed retrofitting system a case study has been analysed. The structure considered in this paper is a symmetrical classical industrial single-storey building with plan dimensions equal to 41.5 m×29 m, completely realized with precast concrete elements assembled with dry connections (Fig. 7). The internal structure is composed by 12 columns with section of 70×50 cm and height equal to 6.95 m. Each column has 28 $\phi 24$ rebars with $\phi 8/15$ cm confinement bars. The roof is simply supported by the columns and it is constituted by precast concrete beams and curved concrete panels. The facades are realized with precast concrete cladding panels of different width, with height equal to 8 m and thickness equal to 20 cm. The longitudinal facades are covered with 15 panels of width equal to 2.77 m, while the transversal facades are cladded with 6 panels of width equal to 2.5 and 2 panels of width equal to 1.49 m. Furthermore, in order to allow the access of materials and machinery of big size, in the transversal directions there are two 5 m wide openings. The loads applied on the structure are the following: dead load due to the roof elements $G_{k,roof} = 1.39 kN/m^2$, snow load $Q_{k,snow} = 0.6\ kN/m^2$, dead load due to the cladding panels in the longitudinal direction $G_{k,panels,x} = 1508\ kN$, dead load due to the cladding panels in the transversal direction $G_{k,panels,y} = 880\ kN$. The structural elements have been designed respecting the procedures provided by the Eurocodes.

Results of Push-over analyses

The performance of the proposed system under seismic events has been verified by means of SAP 2000 computer out modeling the columns with fibre plastic hinges where the program. In particular, the structural model has been carried Takeda and Kinematic plastic models have been adopted to simulate the plasticity arising in concrete and rebars, respectively. The elastic behaviour of structural members is accounted by beam-column elements. In the case of unreinforced structure, external walls are considered only as concentrated masses in the perimeter of the building nodes. In fact, they do not contribute to the stiffness and strength of the structure but only as gravity loads. Push-over

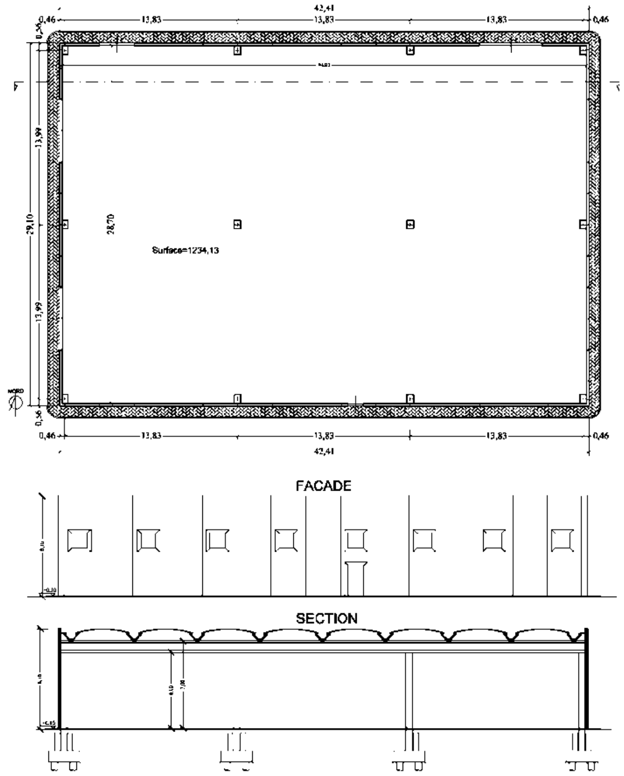


Fig. 7 Plan view, facade and section of the analysed building

Table 1 Results Pushover Analysis

Direction	Pushover Analyses					
	V_1 (KN)	V_y (KN)	μ	R_Ω	R_μ	q
x	1558,67	1668,29	1,52	1,070	1,43	1,53
y	2366,06	2542,38	1,48	1,075	1,40	1,50

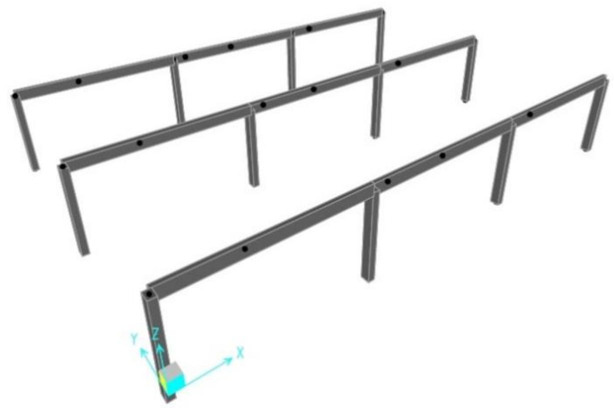


Fig. 8 Structural scheme of the unreinforced structure (Screenshot from SAP2000)

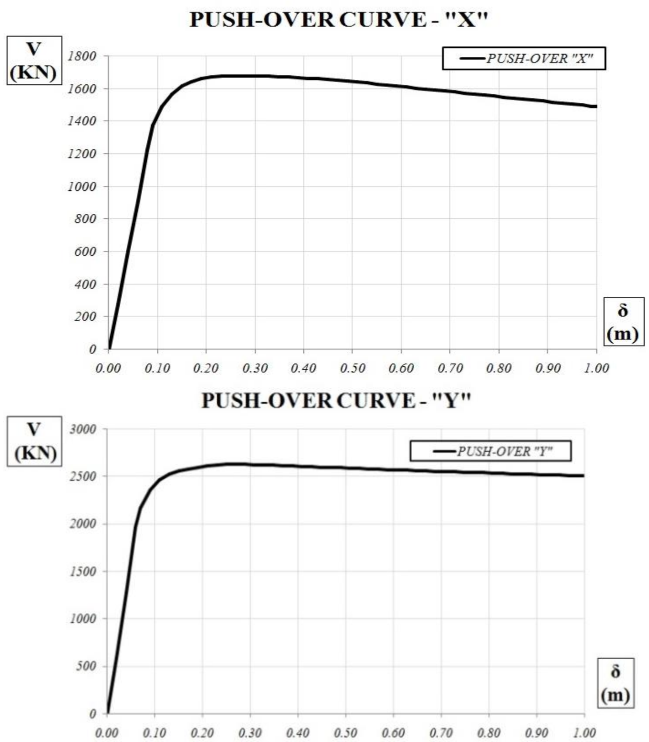


Fig. 9 Push-over curves of unreinforced structure for *X* and *Y* direction

analyses are performed for both x and y direction (Fig. 9) in displacement control taking in account both geometrical and mechanical non linearities by means of an unique load applied in the centre of gravity of the masses of the deck assumed as perfectly rigid (Fig. 8). The performances of the existing structure is evaluated in terms of behaviour factor q

$$q = \frac{V_{eu}}{V_1} = \frac{V_{eu}}{V_y}\frac{V_y}{V_1} = R_\mu \cdot R_\Omega \tag{1}$$

where V_{eu} is the elastic base shear; V_y is the yield shear corresponding to first yielding; V_1 is the base shear corresponding to the first plastic hinge development; R_μ and R_Ω are the ductility factor and the ratio between the shear yield strength of the elastic-plastic system and the analogous one corresponding to the formation of the first plastic hinge (Table 1), respectively.

The ductility μ is calculated as the ratio between the ultimate displacement δ_u, and the displacement occurring at the yield strength of the elastic-plastic system (Table 1)

$$\mu = \delta_u/\delta_1 \tag{2}$$

Results of Dynamic analyses

Incremental Dynamic Analyses have been carried out on the same structural scheme adopted for push-over analyses, for increasing values of the spectral acceleration, on a set of seven accelerograms selected from PEER database for approximatively match the Eurocode 8 design spectrum for soil A and PGA of about 0.2 g once scaled corresponding to the first period of vibration of the structure (T_1=0,493 in x direction and T_1=0,386 in y direction). The period of vibration refers to the cracked section where the cracking has been taken in account by introducing an elastic modulus of the concrete reduced of the 50% as suggested by codes and a Poisson ratio equal to 0. In addition, concrete and steel of columns have been modelled by means of a Takeda and Kinematic model, respectively. In this case, the behaviour factor has been evaluated by means of the following relation

$$q = \alpha_u / \alpha_1 \tag{3}$$

where α_u is the acceleration multiplier at the collapse and α_1 is the acceleration multiplier at first yield. In Table 2, behaviour factors obtained from the dynamic analyses are reported. The behaviour factor assumed for both the direction is the minimum among those provided by all the dynamic analyses performed in the same direction.

Table 2 Behaviour factors for *X* and *Y* directions of unreinforced structure

Earthquake	Direction *x*	Direction *y*
	q_α	q_α
Imperial valley	1,54	1,28
Kobe	1,56	2,59
Northridge	1,83	1,38
Spitak Armenia	2,39	1,72
Victoria Mexico	3,05	4,00
Santa Barbara	1,74	2,00
Friuli Tolmezzo	2,24	1,91
$q_{\alpha,\mathrm{MIN}}$	1,54	1,28

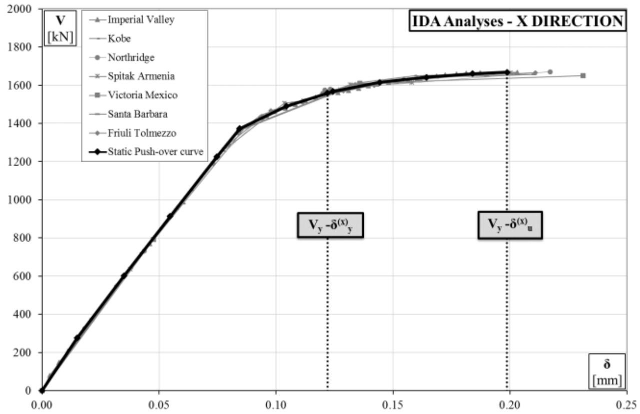


Fig. 10 Static push-over and Dynamic push-overs (*X* direction)

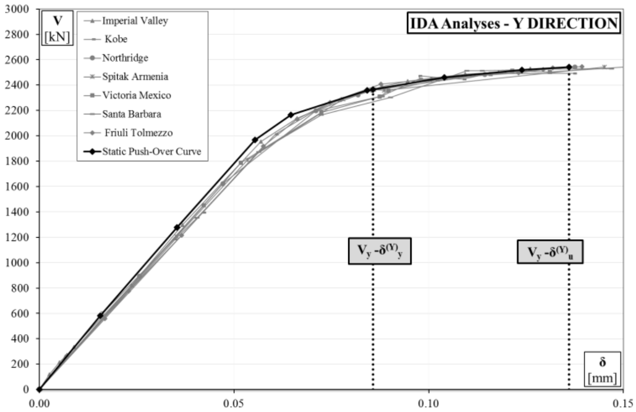


Fig. 11 Static push-over and Dynamic push-overs (Y direction)

In addition, to check the accuracy of IDA analyses a comparison between dynamic push-over curves and static push-over curves is reported in Fig. 10 and Fig. 11 for X and Y direction, respectively. The accuracy of the IDA analysis is confirmed because dynamic push-over curves are in perfect agreement with static push-over ones in both the directions. Dynamic push-overs are the curve provided by the connections of the points identifying the couple of maximum shear-displacement, achieved for each value of the spectral acceleration, opportunely increased. Finally, behaviour factors of about 1.5 and 1.3 for X and Y direction, respectively, have been confirmed and are also comparable with those provided by push-over curves as reported in Table 1.

3.2 Retrofitted structure

As regards the retrofitted structure two XL-stub have been located at the bottom of each external walls in order to provide by hysteresis additional dissipation. These hysteretic devices have to be designed to oppose the actions transferred by the connections located at the top of the walls and to dissipate the seismic energy by means of hysteresis loops activated by the alternate rocking motion of the cladding panels during a seismic event (Latour *et al.* 2015).

Results of Push-over analyses

The SAP2000 structural model of retrofitted structure (Fig. 12) differs from the unreinforced one only for the inclusion of external walls. They are modelled as rigid panels connected, at the top, to the perimeter beams by means of elastic gap links and on the bottom by means of plastic springs in force-elongation. Their constitutive model has been calibrated on the bases of experimental tests. In particular, for push-over curves a trilinear plastic model has been assumed to best fit the monotonic behaviour of the XL-Stub (Fig. 6). Finally, the outcome of push-over analyses, i.e., the behaviour factor have been evaluated, as the same as preliminarily reported, for the unreinforced structure by means of Eq. (1) where

$$R_\Omega = V_{u.L-Stub} / V_{y.L-Stub} \tag{4}$$

$$\mu = \theta_{u.L-Stub} / \theta_{y.L-Stub}. \tag{5}$$

$$R_\mu = (2\mu - 1)^{0.5} \text{ for } T^* < T_c \tag{6}$$

and $V_{u.L-Stub}$ is the ultimate shear of the first XL-stub collapsing, $V_{y.L-Stub}$ is the first yield shear of the first XL-stub involved in plastic range, $\theta_{u.L-Stub}$ and $\theta_{y.L-Stub}$ the plastic rotation evaluated in correspondence of the collapse and first yielding of XL-stub, respectively. Results of push-over analyses on the retrofitted structures and the values of R_Ω, μ, R_μ and q are reported in Fig. 13 and Fig. 14 for *X* and *Y* direction, respectively.

Results of Dynamic analyses

IDA analyses have been carried out on the same structural scheme adopted for the push-over analyses of the retrofitted structure. Only devices modelling need more exhaustive considerations.

In fact, the hysteretic behaviour has been modelled by means of force-elongation plastic springs accounting for the Takeda model calibrated on the basis of the cyclic experimental tests (Fig. 15). Starting from the experimental results, the number of partial cycles and the value of the elongation for each partial and complete cycle has been evaluated to provide the load history of the devices. In order

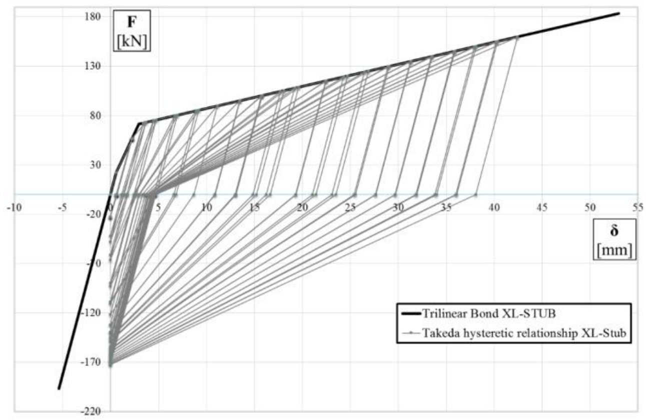


Fig. 15 Analytic cyclic behavior of XL-stub force-elongation hysteretic relationship

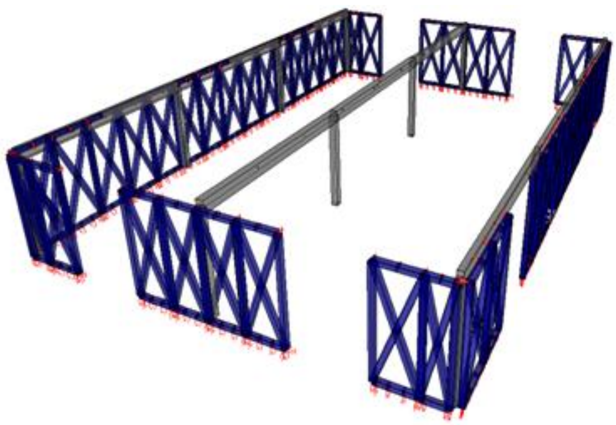


Fig. 12 Structural scheme of the retrofitted structure (Screenshot from SAP2000)

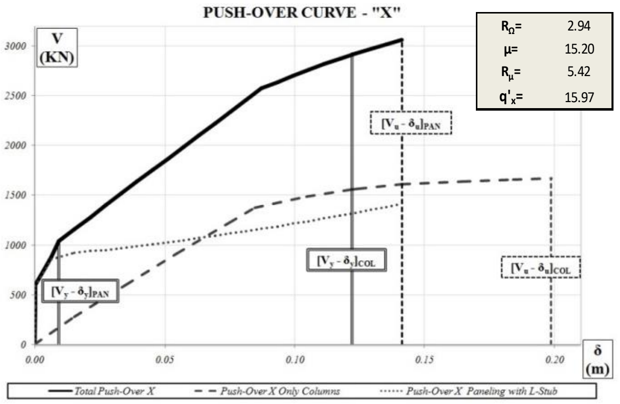


Fig. 13 Push- over curves for X direction (retrofitted structure)

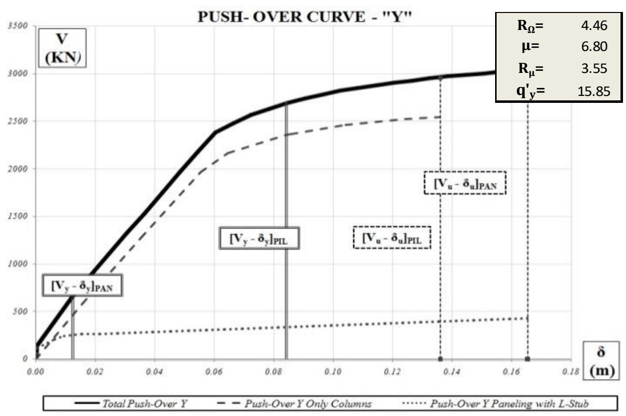


Fig. 14 Push- over curves for Y direction (retrofitted structure)

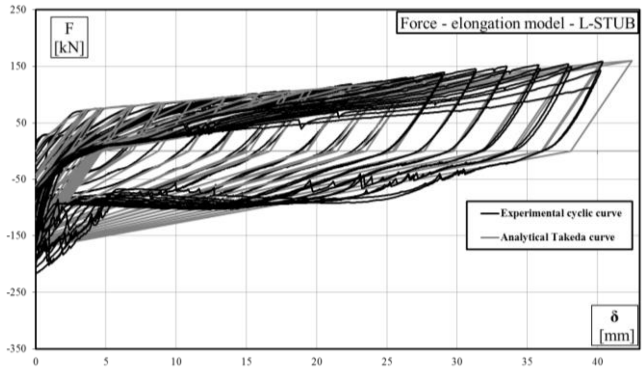


Fig. 16 Calibration of XL-stub cyclic force-elongation behavior

to represent the analytic cyclic behaviour a starting pivot point has been fixed. Its position follows a straight line, with the same shape of the elastic branch of monotonic curve in perfect agreement with the Takeda model. In addition, for the evaluation of the pivot point an energetic approach has been exploited by minimizing the sum of cumulated energy of cycles.

The model provided in this way has been considered suitable to accurately represent the hysteretic behavior of the devices located at the bottom of the external walls as testified by Fig. 16 where the hysteretic response of the XL-stub modelled in SAP2000 is reported and compared with the experimental one.

The behaviour factors in both the direction have been computed according to Eq. (3) where α_u is the minimum among those computed in correspondence of the first column collapse and first XL-stub collapse

$$\alpha_u = min\{\alpha_{u.col}; \alpha_{u.L-stub}\} \tag{7}$$

while α_y is computed in correspondence with the first yielding of XL-stub. In addition, the comparison between dynamic push-over curves and static push-over ones is reported in Fig. 17 and Fig. 18 for X and Y direction, respectively.

The accuracy of the IDA analysis is confirmed because dynamic push-over curves are in perfect agreement with static push-over ones for both the directions. Finally, a behaviour factor of about 2.93 and 6.50 for *X* and *Y* direction are provided. This results appears to be not in agreement with those provided by push-over analyses on retrofitted structure (Fig. 13 and Fig. 14) but are out of dubs more accurate and realistic; firstly, because they belong to a more accurate analysis methodology (IDA analyses) and, secondly, because they are more comparable with the behaviour factors usually reported in seismic codes for high dissipative structures. However, a more extensive analysis should be conducted to provide more general results.

4. Conclusions

In this paper, an innovative approach for the seismic retrofitting of precast concrete structures has been presented. The proposed system, as an alternative to the classical approach, provides to connect rigidly the cladding walls to the internal columns and to introduce at the base of the panels metallic hysteretic dampers in order to obtain a dual system. The design philosophy of the proposed approach has been validated by developing a case study of a single-storey industrial building. In this application the roof has been considered as a rigid diaphragm connecting the columns and, therefore, it is only devoted to structure typologies with rigid slab on the top roof. Alternatively, in the framework of the retrofit the rigid slab should be realized with bracing systems.

Table 3 Behaviour factors for *X* and *Y* directions of retrofitted structure

Earthquake	Direction *x*	Direction *y*
	q_α	q_α
Imperial valley	4,75	12,68
Kobe	14,86	11,43
Northridge	14,81	10,00
Spitak Armenia	2,93	6,50
Victoria Mexico	3,00	6,60
Santa Barbara	10,00	6,60
Friuli Tolmezzo	14,57	20,00
$q_{\alpha,\mathrm{MIN}}$	2,93	6,50

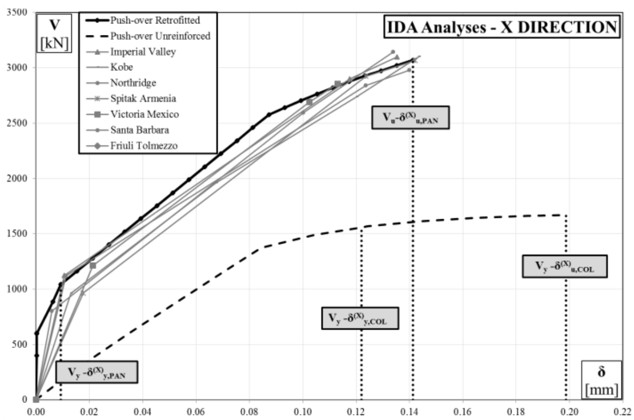


Fig. 17 Static push-over and dynamic push-overs (*X* direction)

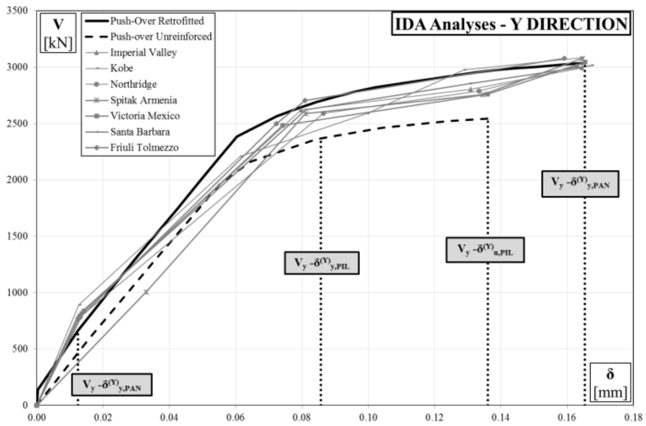


Fig. 18 Static push-over and dynamic push-overs (*Y* direction)

The results of the pushover and IDA analyses have pointed out that the dual system can be easily modelled as the assemblage in parallel of the system of columns and of the cladding panels. In addition, following the proposed design criterion it has been pointed out that, if properly designed, the system is able to develop a significant over-strength and ductility, testified by high values of behaviour factor, protecting, in the same time, the internal columns from any plasticization. Finally, a preliminary evaluation of the behaviour factor of the structure is also reported. However, the results herein proposed are limited to one building only and need to be extended to a large number of cases in order to provide more robust conclusions.

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