Seismic Response Comparisons between RC Precast Structures with Dissipation Devices on Beam-Column Connections

S. Santagati, D. Bellotti, D. Bolognini
European Centre for Training and Research in Earthquake Engineering, Pavia, Italy

E. Brunesi
ROSE Programme, UME School, IUSS Pavia, Italy

SUMMARY:
The majority of the Italian industrial one-storey and multi-storey buildings consists of reinforced concrete precast structures with monolithic columns and pin-ended beams, potentially characterized by high flexibility and low resistance capacity of the beam-column and panel-structure connections.
In this paper, different typical Italian precast typologies of buildings up to 5 stories are designed using the Forced Based approach suggested by the current Italian seismic code.
Secondly, innovative typologies of beam-column connections, developed to improve the seismic performance of existing and new industrial and residential precast buildings have been designed following both a Displacement-Based procedure and a Force-Based one.
Finally, the effectiveness and the possible advantages of both new solutions and alternative structural typologies have been evaluated.

Keywords: precast structures, connections, seismic perimeter walls, viscous devices, friction devices

1. INTRODUCTION

The majority of the Italian industrial one-storey and multi-storey buildings consists of reinforced concrete precast structures with monolithic columns and pin-ended beams, potentially characterized by high flexibility and low resistance capacity of the beam-column and panel-structure connections. For these reasons, it is generally accepted that the seismic response of other structural precast typologies (e.g. perimeter walls, re-centring post-tensioned connections, equivalent monolithic structures) is more appropriate.
Some solutions, typically used for cast-in-place concrete structures may be ineffective for several precast structure configurations (e.g. braced systems, due to the relevant heights and span lengths or base-isolation, due to the high flexibility). Alternative solutions based on the use of energy dissipation devices at the beam-column joints, in order to reduce the displacement demand avoiding an excessive increase of local force demand, have been examined in the past on RC structures [Dolce et al, 2007] and on particular typologies of RC precast structures [Priestley et al, 1999; Kurama, 2000; Holden et al, 2003; Morgen and Kurama, 2008; Martinelli and Mulas, 2010].
In this paper, different typical Italian precast typologies of buildings up to 5 stories, as well as alternative typologies (e.g. perimeter walls), are designed using the Forced Based approach suggested by the current Italian seismic code. Secondly, innovative typologies of beam-column connections, developed to improve the seismic performance of existing and new industrial and residential precast buildings have been designed following both a Displacement-Based procedure and a Force-Based one. The new connection solutions can be classified as partially fixed connections, beam-column joints with devices dissipating energy by friction or with viscous-elastic dampers, and a combination of partially fixed connections and viscous-elastic dampers at the beam-column joints.
Finally, the characteristics of the response of the new buildings designed in accordance with the current Italian code and the effectiveness and the possible advantages of both new solutions and alternative structural typologies have been evaluated.

2. SEISMIC DESIGN OF TYPICAL ITALIAN PRECAST STRUCTURES

Generally, the market of precast buildings is strongly affected, in Italy, by the speed in construction, to the detriment of the detailing of the reinforcement and of the effectiveness of the connections in seismic zones. In the case of low-rise buildings of not primary importance, the precasters prefer to use dry connections able to transmit only the horizontal shear action, or, in some cases of multi-storey building, moment resisting connections, whose characteristics in terms of seismic response (especially the degradation of the seismic performances due to the cyclic loads and the effectiveness of the anchorages of the reinforcing bars) are not known.
In the majority of the cases, since the speed in construction and the economical aspect are the only parameters which govern the choice between a cast-in-place or a precast structure, the solution considered by the precasters in order to improve the seismic response of the flexible buildings is to adopt large sections of the columns (even 1000x1000mm in the case of three-storey buildings). It is evident that the cost of the post-earthquake repairing interventions are not considered and, in addition, due to the extremely high competitiveness, often the designers interpret the code provisions in a very particular and personal way, in order to favor convenient details and solutions, not always seismically efficient. This approach has been used in this work to design flexible one-storey and multi-storey reinforced concrete precast structures with pin-ended beams and monolithic columns in accordance with the current Italian code. The following main results have been obtained:

1) The second order effects and the flexibility of the examined structures (one to five stories) govern the design (Figure 2.1a).
2) Larger sections of the columns, adopted to limit the second order effects and the displacements at the damage limit state, resulted in an increment in strength, often due to the minimum longitudinal and transversal reinforcement requirements. In particular the minimum amount of transversal reinforcement depends on the depth and width of the column section, generally resulting in an extremely low spacings between the stirrups (e.g. < 50 mm).
3) The increased strength results in a reduced effective displacement ductility or even in an elastic response.
4) The connections, if not designed accordingly to the “modified” structural response (equivalent to the reduction of the behavior factor depicted in figure 2.1b), may be the weak point of the system, because they may activate an anticipated, not dissipative, collapse mechanisms.
5) The foundation system depends on the strength of the columns both in high and low ductility class, so over-designed columns result in over-designed foundations.

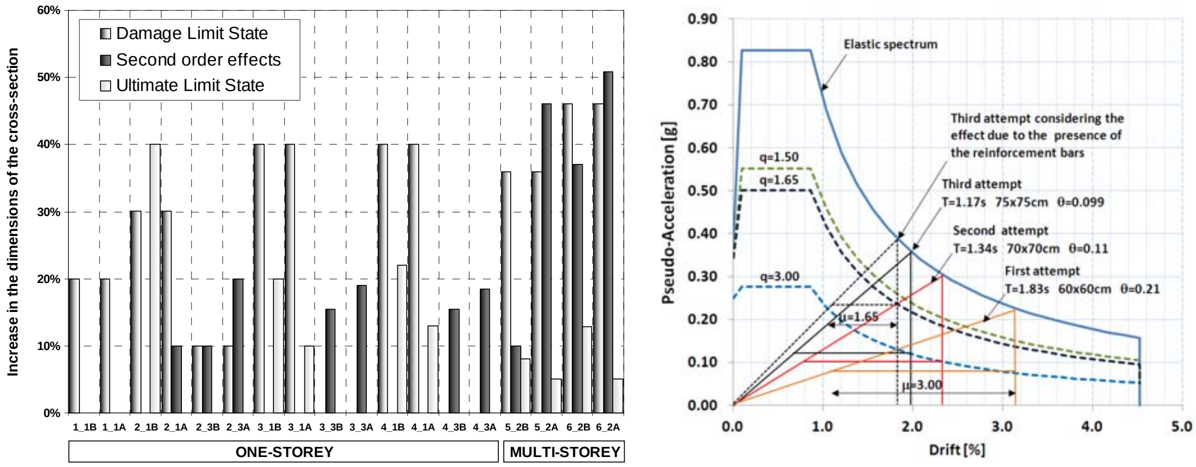


Figure 2.1 New designed structures with pinned beams and monolithic columns: increment of the section of the column to limit the second order effects and the excessive displacements at the damage limit state (left); stiffness increment and subsequent reduction of the behaviour factor q (from 3 to 1.65 in the case depicted in figure) necessary to limit the second order effects (right).

Finally, two further issues have to be highlighted: a) the real constraint at the base of the columns when precast pocket foundations are used (depending on the characteristics of the filling material, the column may rotate into the pocket foundation up to a drift of 0.04%); b) the expensiveness of a design developed in high ductility class, rather than in low ductility one, in the case of the examined typologies, especially for multistory buildings in medium-low seismicity zones (e.g. PGA < 0.20 g).

3.DISSIPATION DEVICES OF ONE-STOREY AND MULTISTOREY BUILDINGS

In the second step of the work, possible retrofitting solutions were proposed. Among them, the solutions based on external devices to be placed at the beam-column joints of buildings up to 5 stories have been developed. Three basic devices (energy dissipated by friction, viscous-elastic devices, diagonal elastic springs which increase the stiffness of the structure and provide re-centering capacities) and different their combinations were considered. The buildings were designed in accordance with the following two criteria: a) structures up to 5 stories according to the Italian seismic code provisions; b) structures with slender columns, initially not in accordance with the code, due to the excessive high flexibility. The external devices were used in combination with the second design criterion in order to satisfy the code requirements. The considered case-studies are summarized in the following table 3.1.

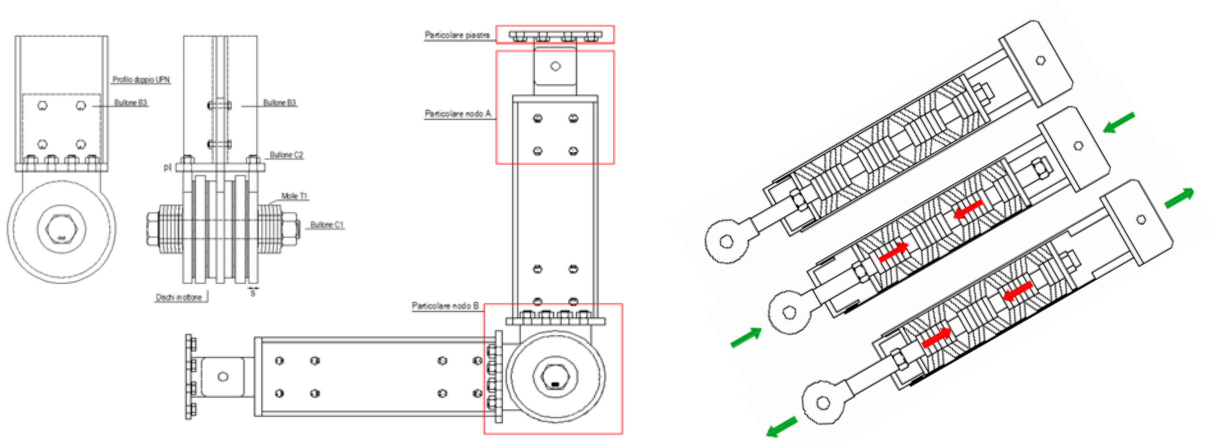
Table 3.1. Dissipation devices, viscous-elastic devices and diagonal elastic springs: summary of the examined case-studies (M_D is the moment of activation of the devices which dissipate energy by friction)

<table>
<tr><th>ID</th><th>Type of retrofitting</th><th>Distribution of devices in the seismic resisting frame</th><th>Structural typology</th><th>Type of analysis</th></tr>
<tr><td rowspan="2">1</td><td rowspan="2">1 disk dissipating energy by friction M_D = 40, 80, 120 kN</td><td>At each connection</td><td rowspan="7">3-4-5-storey structures with pinned beams
One storey structures with pinned beams</td><td rowspan="10">Time history analyses with 7 accelerograms</td></tr>
<tr><td>Only in the internal connections</td></tr>
<tr><td>2</td><td>3 disks dissipating energy by friction M_D = 40, 80, 120 kN</td><td>At each connection</td></tr>
<tr><td>3</td><td>Without devices</td><td>-</td></tr>
<tr><td>4</td><td>Diagonal springs</td><td>At each connection</td></tr>
<tr><td>5</td><td>Diagonal springs + Devices dissipating energy by friction</td><td>At each connection</td></tr>
<tr><td>6</td><td rowspan="3">Diagonal viscous-elastic devices</td><td rowspan="2">At each connection</td></tr>
<tr><td>7</td><td rowspan="3">3-4-5-storey structures with hinged external connections and monolithic internal ones</td></tr>
<tr><td>8</td><td>Only at each hinge</td></tr>
<tr><td>9</td><td>Without devices</td><td>-</td></tr>
</table>

The work consisted of evaluations, through parametrical analyses on buildings up to 5 stories, of the more appropriate characteristics to assign to the external devices. The details of each devices, then, have been designed, except for the viscous-elastic devices. In particular:

- A method for the detailed design of the components of systems up to 3 disks, dissipating energy by fiction, characterized by at least two rotating surfaces of steel and brass subjected to different levels of post-tension (figure 3.1a), has been developed. The main design parameters, considered in order to evaluate the performance of the device in terms of energy dissipative capacity and load reduction on the columns, are: configuration, dimensions and number of disks, number of rotating contact surfaces, post-tension level, materials of surfaces in contact.
- A method for the detailed design of the components of diagonal linear springs, able to provide to the structure a stiffness increment and re-centering capacity (figure 3.1b), has been developed. This device consists of an external rigid cylinder and an internal system of series of packs of special springs connected in parallel in order to give an appropriate level of

deformation capacity. The dimensions of the external cylinder, the post-tension level, number of packs of springs, number of springs, typology of springs, as well as the relative displacement capacity are the main parameters which have to be calibrated in the design process.



(a) (b)

Figure 3.1 Details of the device dissipating energy by friction made of one disk with 4 rotating surfaces (left) and diagonal elastic spring devices (right).

The successive step was the development of a method to calculate the interaction between the devices and the structure, since the presence of the device affects the global stiffness of the structure and the design load distribution. The stiffness of the system is k = (Num / Den), where Num and Den are explained below for each considered configuration of the devices and J_1, J_2 are the moment of inertia of the column and of the beam, E is the corresponding elastic Young modulus, E_3J_3 is the stiffness of the device, H and L the height and the span length of the frame, finally $X=E/E_3$. Also the moment M_A at the base of the column where the device is placed, is M_A = (Num / Den).

Friction devices: stiffness of the system (eq. 3.1) and moment at the base of the columns (eq. 3.2):

$$Num = 12EJ_1\begin{bmatrix} 4b^2J_1J_3 + \\ -4b(-J_1J_2X + J_1J_3 + J_2J_3)L + \\ +J_3L(6J_2H + J_1L) \end{bmatrix} \qquad Den = \begin{bmatrix} -b^4J_2J_3L - 8bH^3(-J_1J_2X + J_1J_3 + J_2J_3)L + \\ +J_3H^3L(3J_2H + 2J_1L) + \\ +2b^2J_3H^2(4J_1H + 3J_2L) \end{bmatrix} \tag{3.1}$$

$$Num = F\begin{bmatrix} -4bH(-J_1J_2X + J_1J_3 + J_2J_3)L + \\ +J_3HL(3J_2H + J_1L) + \\ +b^2J_3(4J_1H + J_2L) \end{bmatrix} \qquad Den = 2\begin{bmatrix} (4b^2J_1J_3 - 4b(-J_1J_2X + J_1J_3 + J_2J_3)L + \\ +J_3L(6J_2H + J_1L)) \end{bmatrix} \tag{3.2}$$

Diagonal springs: stiffness of the system (eq. 3.3) and moment at the base of the columns (eq. 3.4):

$$Num = 12EJ_1\begin{bmatrix} 12EJ_1J_2L + \\ +k_{dis}b^2(4b^2J_1 - 4b(J_1 + J_2)L + \\ +(6J_2H + J_1L)L) \end{bmatrix} \qquad Den = \begin{bmatrix} 24EJ_1J_2H^3L - k_{dis}b^6J_2L + \\ -8k_{dis}b^3H^3(J_1 + J_2)L + \\ +k_{dis}b^2H^3L(3J_2h + 2J_1L) + \\ +2k_{dis}b^4h^2(4J_1h + 3J_2L) \end{bmatrix} \tag{3.3}$$

$$Num = \begin{bmatrix} \dfrac{12FH}{k_{dis}b}EJ_1J_2 + Fb^2J_2 2 - 4FbHJ_2 + \\ +3FH^2J_2 - 4FbHJ_1 + \\ +\dfrac{4Fb^2H}{L}J_1 + FHLJ_1 \end{bmatrix} \qquad Den = \begin{bmatrix} \dfrac{24}{k_{dis}b}EJ_1J_2 - 8bJ_2 + 12HJ_2 + \\ -8bJ_1 + \dfrac{8b^2}{L}J_1 + 2LJ_1 \end{bmatrix} \tag{3.4}$$

Combination of the two devices, global stiffness (eq. 3.5) and moment at the base (eq. 3.6):

$$Num = 12EJ_1\begin{bmatrix} 4b^4k_{dis}J_1J_3 + \\ +b^2k_{dis}J_3L(6J_2H+J_1L)+ \\ +12EJ_1J_2J_3L(-1+\alpha)^2 + \\ 4b^3k_{dis}(XJ_1J_2\alpha^2 - (J_1+J_2)\ J_3) \end{bmatrix} \qquad Den = \begin{bmatrix} 8H^3b^4k_{dis}J_1J_3 + \\ +2H^3b^2k_{dis}J_3L(6J_2H+J_1L)+ \\ +24H^3EJ_1J_2J_3L(-1+\alpha)^2 + \\ +8H^3b^3k_{dis}(XJ_1J_2\alpha^2 - (J_1+J_2)\ J_3)+ \\ -b^2k_{dis}J_2J_3(b^2-3H^2)^2L \end{bmatrix} \tag{3.5}$$

$$Num = FH\begin{bmatrix} \frac{12E}{k_{dis}b}+\frac{b^3}{J_1H}-\frac{4b^2}{J_1}+\frac{3Hb}{J_1}+ \\ -\frac{4b^2}{J_2}+\frac{4b^3}{J_2L}+\frac{Lb}{J_2}-\frac{24\alpha E}{k_{dis}b}+ \\ +\frac{12\alpha^2E}{k_{dis}b}+\frac{4b^2\alpha^2X}{J_3} \end{bmatrix} \qquad Den = \begin{bmatrix} \frac{24E}{k_{dis}b}-\frac{8b^2}{J_1}+\frac{12Hb}{J_1}-\frac{8b^2}{J_2}+\frac{8b^3}{J_2L}+ \\ +\frac{2Lb}{J_2}-\frac{48E\alpha}{k_{dis}b}+\frac{24E\alpha^2}{k_{dis}b}+\frac{8Xb^2\alpha^2}{J_3} \end{bmatrix} \tag{3.6}$$

Where $\alpha = \frac{3\sqrt{2}J_3}{A_{dis}b^2+3\sqrt{2}J_3}$, A_{dis} = area of the disk dissipating energy by friction

The calibration of the optimal parameters of the friction devices, of the linear springs or of the combinations of these two devices, which give an appropriate seismic response to the structures, has been based on parametric time-history analyses with 7 natural accelerograms (figures 3.2, 3.3, 3.4).
At the end of this step, the details of the devices are designed in accordance with the method partially described at the beginning of this chapter. The reinforced concrete elements (beams and columns) are then designed imposing the hierarchy of strength method. The devices must activate before the columns reach the yielding condition, in order to avoid a non-linear response. The last check consists of new time-history analyses with the final characteristics of the devices and of the reinforced concrete elements, in order to prove the good seismic response of the structure in terms of local and global behavior.
Structures with pinned beams and viscous-elastic dampers or with partially fixed connections and viscous-elastic dampers have been also considered in this work. The design method is based on the approach used by Christopoulos & Filiatrault (2006). Within this method, an equivalent MDOF model, made of one mass and two non-linear spring elements for each floor, has been calibrated in order to perform quick parametric time-history analyses. The final check consists of time-history analyses on the global model of the structure. The behavior of the springs of the equivalent MDOF model has been calibrated on the results obtained from the characteristics of the response of a reference global structure. In particular, the calculation of the distribution of the initial stiffness is based on a corrective factor calibrated in order to match the most important periods and modes of of vibration of the global structures. The hysteretic behavior of the springs has been calibrated, in a simplified way, on the base of the non-linear response of the global structure. Since the method does not consider the effects due to the vertical loads which are generated from the horizontal seismic loads, it is necessary to define appropriate coefficients, one for each floor, in order to modify the stiffness distribution along the height of the structure.
The results of the design and of the time-history analyses are briefly summarized:

- All examined solutions were effective in the limitation of the displacement at the ultimate limit states and partially at the damage limit state; the use of the devices permit to save the 45% of material of the columns.
- Some solutions (diagonal springs, combination of friction devices and diagonal springs) are only partially effective. Since they generate an increment of the local loads greater than 50%, particularly at the joint between the column and the beam. The use of the friction devices is more effective, but the increment of the shear action at the connection is included in the range 10%÷55%, so they are not useful in the case of existing structures.
- The viscous-elastic dampers are very effective and permit to save more than 55% of material (concrete and steel of the columns).

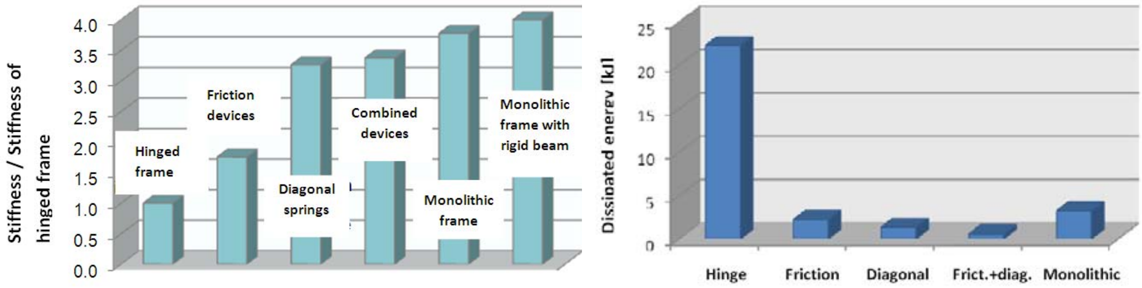


Figure 3.2 Stiffness of the system (left) and energy dissipated by the column (right) as a function of the typology of considered device.

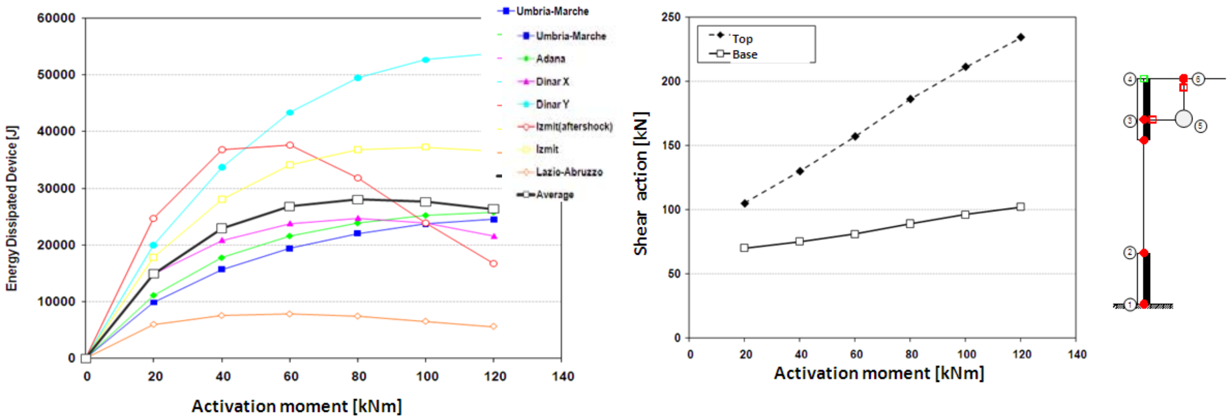


Figure 3.3 Friction devices: energy dissipated by the device as a function of the moment of activation (the average values are depicted with the black line, left) and top and base shear on the column (right).

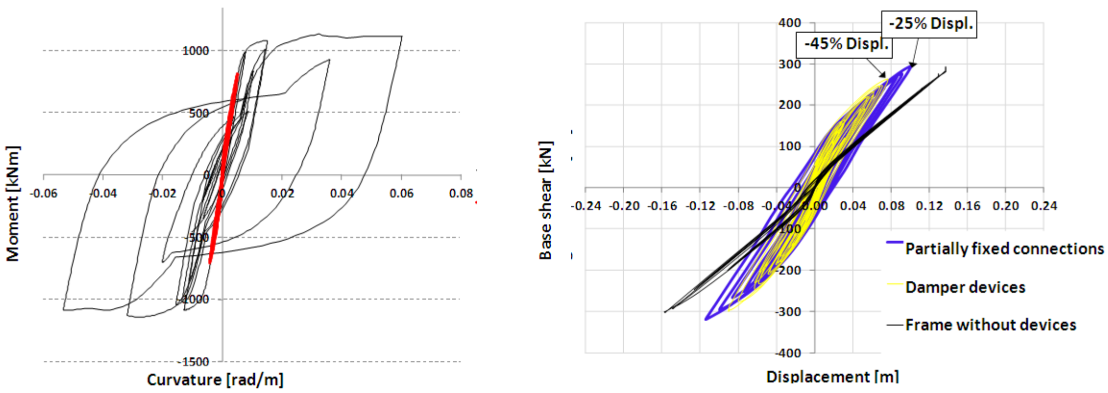


Figure 3.4 Moment-Curvature at the base of a one-storey structure with and without friction devices (red and black lines, left) and top displacement vs. base shear curves of one-storey structure with and without devices (black line: hinged frame; blue line: partially fixed connections; yellow line: viscous dampers, right).

4. PARTIALLY FIXED CONNECTIONS AND EXPERIMENTAL TESTS ON BEAM-COLUMN CONNECTIONS

Since the research was mainly addressed to common and very diffused structures, included structures of not primarily importance, it seemed important to develop a simple solution which may reduce the excessive flexibility and provide an appropriate effective dissipation, able to instantaneously activates in order to limit the displacements and satisfy the requirements at the damage limit state. For these reasons a system of partially fixed connections were defined and developed (figure 4.1). The connection consists of longitudinal steel bars protruding from the beam and anchored into the column through a system of holes and grouting with high resistance mortar, which also fills a void of 10cm between the column and the end of the beam. The beam is supported by a reinforced concrete corbel; a layer of neoprene of 10mm is placed between the beam and the corbel. The protruding steel bars

works along their longitudinal axes and are unbonded within the layer of mortar, in order to limit the damage of the mortar. In the case of internal joints, the bars may be anchored by overlapping or connected with a mechanic coupler, whilst, in the case of exterior joint, the bars may be anchored by a steel plate or even only by an appropriate bonded length.
The results of the experimental tests (figure 4.2) demonstrated that: it is possible to reduce the amount of material of the vertical elements of 40%, the collapse is characterized by a ductile mechanism without strength degradation up to a drift equal to 4%. The flexural resistance is similar to the one expected in case of monolithic reinforced concrete connection, the damage is concentrated only at the interface between the mortar and the beam. The beam and the column are characterized by an elastic response without damage, the equivalent viscous damping is equal to 15% up to drift of 1.2% and 18%-20% for higher drift levels. This solution is a dry connection in accordance with the expectations of the precasters.
The drawbacks of the system are represented by the big crack, which arises at the early stages (drift 0.8%), the not negligible residual displacements, the repairing interventions necessary after the seismic event.

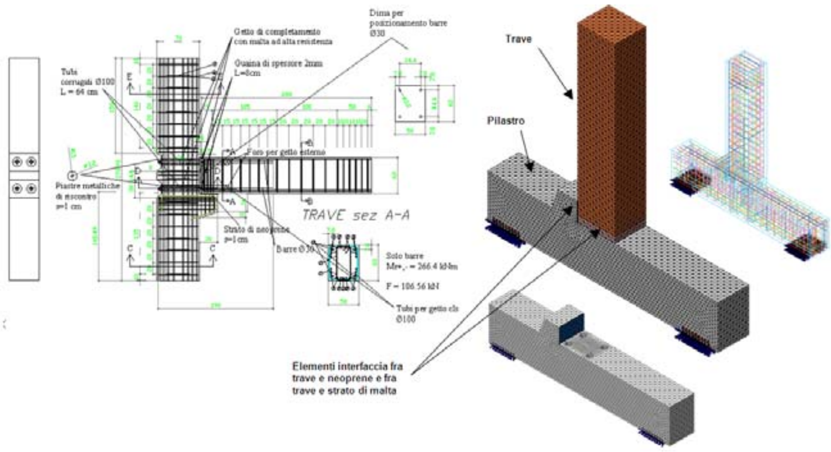


Figure 4.1 Details of the specimen (left) and finite element model developed to evaluate the numerval prediction of the cyclic response of the system of connection (right).

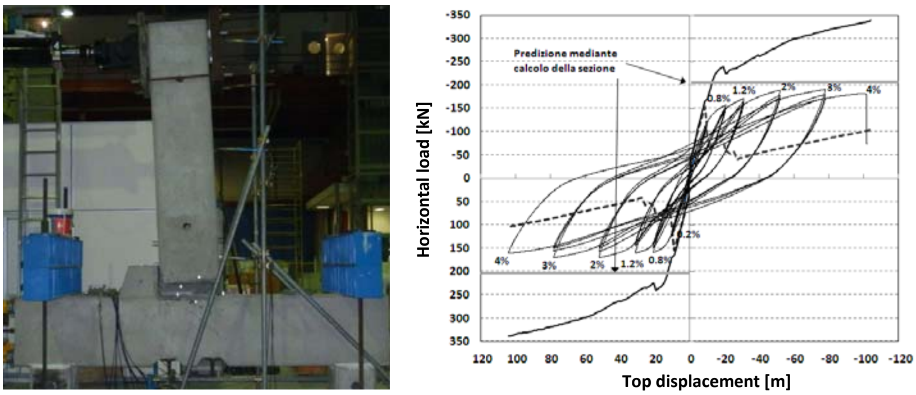


Figure 4.2 Experimental tests: deformed configuration at 3% of drift level (left) and experimental force vs. displacement curves compared to numerical predictions developed under different hypotheses (right).

5. BUILDINGS WITH SEISMIC PERIMETER WALLS AND SLENDER INTERNAL COLUMNS

The structures characterized by perimeter walls are usually seismic efficient to be considered a seismic solution itself. The comparisons with hinged frames highlighted that the perimeter walls, if well designed, resist the seismic loads and limit the second order effects and the flexibility of the structure, while the internal columns resist only the gravity loads. Nonetheless, some requirements of the Italian

seismic code, in particular regarding the effective shear strength and the shear demand, strongly limit the gap between the seismic performances of the two structural solutions.
Four different configurations have been compared: hinged frame, perimeter walls designed in accordance with the hypotheses of low ductility class and two configurations of walls designed in high ductility classes.
Compared to the amount of materials necessary for the hinged frame, the design of perimeter walls developed in accordance with the prescriptions of the Italian code for low ductility class, permitted to save the 15% of concrete and the 40% of steel. The advantages are lower if the design is developed in high ductility class, due to the minimum requirements prescribed by the code (figure 5.1).

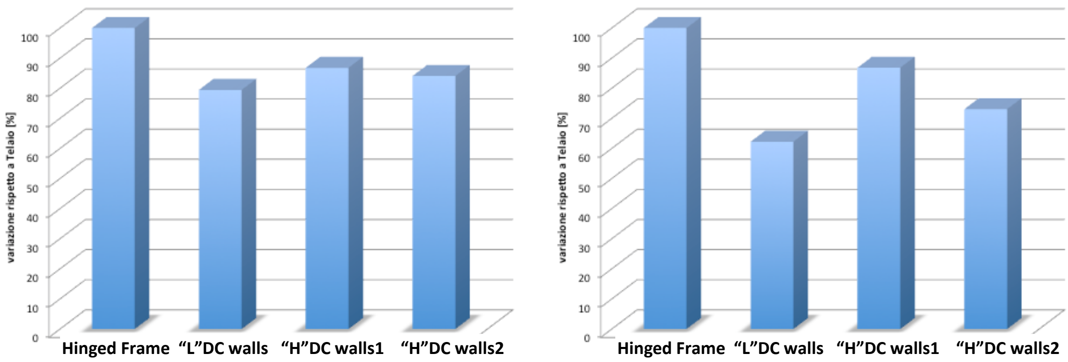


Figure 5.1 Comparisons in terms of amount of concrete (left) and steel bars(right) of the examined case-stuides: from the left to the right: hinged frame, perimeter walls calculated in low ductility class, two different perimeter walls distributions calculated in high ductility class (right).

6. SEISMIC PERIMETER WALLS WITH CONTROLLED MULTI-ROCKING MECHANISM WITH INTERNAL DISSIPATERS

In the previous chapter 6, the structures characterized by perimeter walls have been designed within a force-based method. The same structures have been also studied and designed within a displacement-based approach, characterized by a controlled multi-rocking mechanism with internal dissipaters, developed within the research described in this paper. The method has been resolved by a system of 6 equations, obtained from the development of a multi-modal spring model calibrated on experimental test results (Rahaman & Restrepo, 2000), the evaluation of the variability of 700000 combinations of parameters and the determination of damping vs. ductility curves based on 2000 time-history analyses characterized by different hypotheses (Mpampatsikos, 2009).

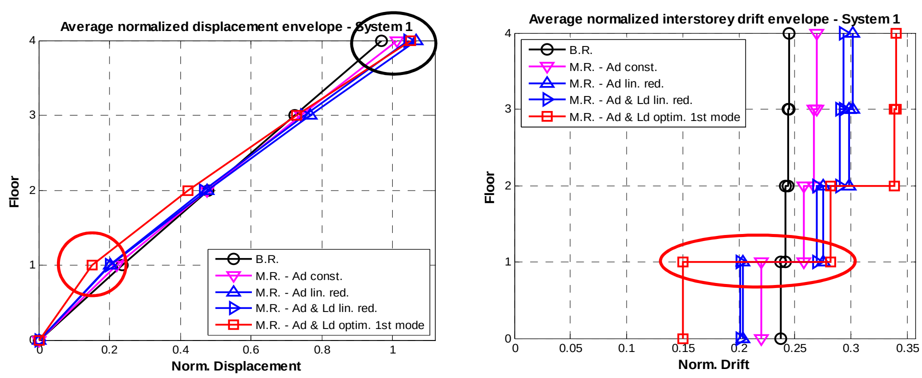


Figure 6.1 Four-storey buildings: average value of the normalized displacement (left) and drift (right).

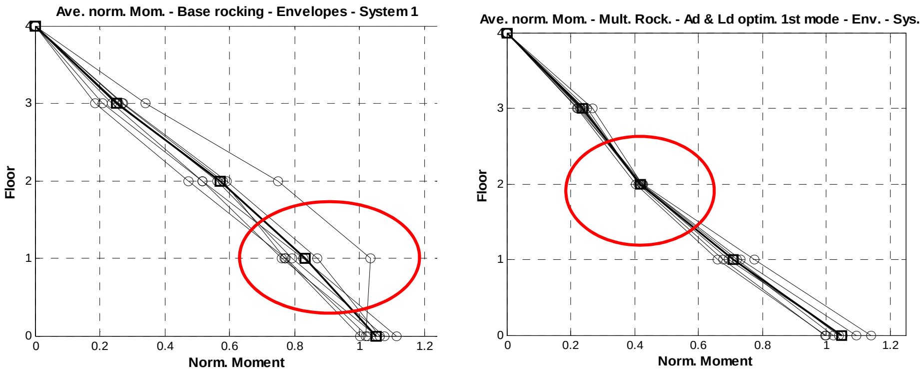


Figure 6.2 Four-storey buildings: normalized moment distribution for base rocking (left) and for dissipaters optimized for first mode of vibration (right).

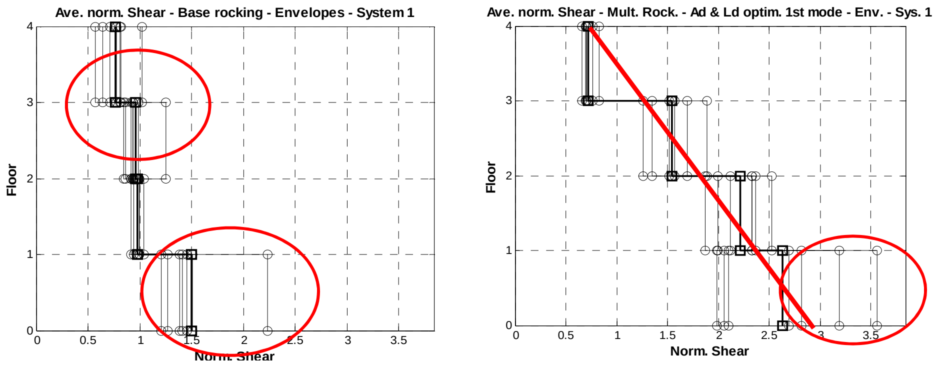


Figure 6.3 Four-storey buildings: normalized shear distribution for base rocking (left) and for dissipaters optimized for first mode of vibration (right).

The reliability of the method has been evaluated by a relevant number of incremental time-history analyses on several case-studies, in order to demonstrate that it is possible to inhibit the effects of the higher modes, to impose the moment distribution along the height of the building and to control the global response both at the damage and the ultimate limit state. Few results of the time-history analyses performed on 4-storey buildings are depicted in figures from 6.1 to 6.3 for different retrofitting techniques based on different distribution of the characteristics of the dissipaters.

7. COMPARISONS AND CONCLUDING REMARKS

Although the interventions based on external energy dissipation devices are strongly effective in case of buildings of relevant importance, some advantages are recognizable also for minor buildings. More in detail, the friction devices give undoubted advantages in terms of global seismic response (figures 3.2, 3.3a, 3.4a). Nonetheless, some small uncertainties still remains in terms of limitation of displacements and deformations at the damage limit state, whilst major uncertainties are related to the increased loads at the beam-column joints, especially for existing structures, and to the geometrical dimensions (rigid arms of 1000 mm, diameter of the disks of 160÷200 mm and total weight of 2÷2.5kN).

Viscous-elastic dampers are very effective and more easy to place. The length of the cylinder (70 ÷80cm), the maximum load (200÷350kN) and α_v equal to 0.4 are characterized by reasonable design values. Although the small stroke demand at the damage limit state may be not consistent with the effectiveness of the devices in order to limit the displacement at the damage limit state, it has to be highlighted that the velocity is the fundamental parameter which characterize the response of the device. Moreover, since dampers need an appropriate level of internal pressure, the devices defined in this works require a minimum diameter of the external cylinder of 200mm.

The structures characterized by perimeter seismic walls are very effective and in particular the design approach based on the controlled multi-rocking mechanism leads to relevant advantages in terms of controlled damage distribution, controlled moment distribution and performance levels. Although in

this work the method has been tested for buildings up to 4 stories, it is effective also for 8 and 12-storey buildings (Mpampatsikos, 2009) and it is very effective if compared with hinged frames, which cannot exceed 2÷3 stories.
Despite its simplicity, the modular system solution made of both hinged and partially fixed connections is effective in the case of the very diffused multi-storey buildings of not primary importance, since its capacity to increase the stiffness of the structure, to satisfy the requirements at the damage limit state and to limit the bending moment at the base of the columns.
Economical comparisons limited to the vertical elements of the structures and to the foundations, show that the structural solutions characterized by perimeter walls and by partially fixed connections represent the most effective typologies (figure 7.1).
The solution characterized by perimeter walls has various advantages: use of precast elements, which can be quickly connected, the internal slender columns and beams require low amounts of concrete and steel, the design of the foundation is not difficult.

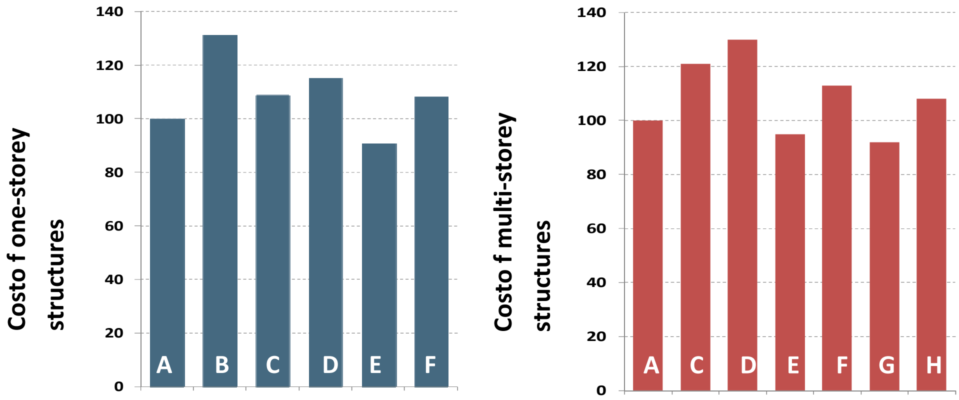


Figure 7.1 Comparison of the costs of the examined structures: A = hinged frames, B = top isolation, C = friction devices, D = viscous-elastic dampers, E = partially fixed connections, F = D + E, G = perimeter walls, H = perimeter walls with controlled multi-rocking mechanism.

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