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Seismic design of a precast r.c. structure equipped with viscous dampers

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Abstract. The seismic design of a two-storey precast reinforced-concrete building structure equipped with viscous dampers is presented in this paper with twofold purpose. The first goal is to verify the applicability of a practical procedure for the identification of the mechanical characteristics of the viscous dampers which allow to achieve target performance levels, originally proposed by the authors for moment-resisting building frames, also with reference to "pendular" structures. The second goal is to investigate the effectiveness of the use of viscous dampers (as compared with traditional lateral-resisting stiff braces) for the seismic design of precast not moment-resisting concrete structures.

Keywords: precast r.c. structure; added viscous dampers; design procedure; seismic response.

1. Introduction

Manufactured viscous dampers (Soong and Dargush 1997, Constantinou *et al*. 1998, Hart and Wong 2000, Christopoulos and Filiatrault 2006) are hydraulic devices characterised by a highly non-linear behaviour, which is commonly modelled using the Maxwell spring-dashpot system which sees a non-linear damper in series with a spring, as represented in Fig. 1. In this figure, F is the force exerted by the damping device, c_{NL} is the damping coefficient, v is the relative velocity between each end of the device, α is the damping exponent, k_{oil} is the stiffness representing the compressibility of the oil in the chamber of the damping device, and x is the damper elongation (or shortening) due to the compressibility of the oil. The axial force developed by a non-linear viscous damper is expressed by

$$F = c_{NL} \cdot \mathrm{sgn}(v) \cdot |v|^{\alpha} \tag{1}$$

where sgn(...) is the sign function. Note that c_{NL} has dimensions of $F \cdot L^{\alpha} \cdot T^{-\alpha}$, with F, L and T representing force, length and time, respectively, and that, for manufactured viscous dampers, the experimentally determined values of the α exponent are usually comprised between 0.15 and 1.0, such values depending on the specific application and on the manufacturer (www.taylordevices.com, www.fip-group.it, www.alga.it).

The success and the wide diffusion of viscous dampers (Hart and Wong 2000, Christopoulos and

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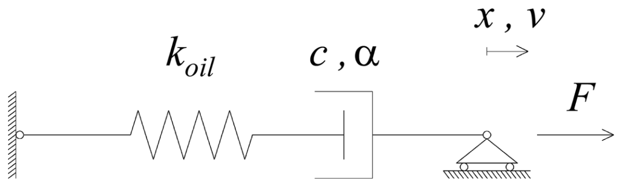


Fig. 1 The Maxwell spring-dashpot system

Filiatrault 2006, www.taylordevices.com, nisee.berkeley.edu, www.arup.com) with respect to other kinds of dissipative devices is mainly due to the following two reasons (Christopoulos and Filiatrault 2006): (i) they are velocity-activated devices, so that the forces generated by linear viscous dampers in a structure are "out-of-phase" with the forces generated by the structural system; (ii) the force in non-linear viscous dampers is ceiled: this allows to avoid overloading the damper and the bracing system to which it is connected due to uncertainties in the seismic excitation and in the structural response.

Since the 1980's added dampers have been the object of several research works. Many of them have become benchmark works in the field. De Silva (1981) developed a gradient algorithm for the optimal locations of discrete passive dampers for flexible systems. Constantinou and Tadjbakhsh (1983) optimised the sizing of a first storey damper. Gürgöze and Müller (1992) studied the optimal damper sizing and placement of a viscous damper for a linear conservative mechanical system on the basis of an energy criterion. Hahn and Sathiavageeswaran (1992) dealt with the effects of changes in the distribution of added viscoelastic dampers on the seismic response of short and tall shear building models. Zhang and Soong (1992) proposed a sequential procedure based on the concept of degree of controllability for optimally placing viscoelastic dampers to shear-type structures. Gluck *et al*. (1996) presented a method of design of supplemental passive damping devices based on optimal linear control theory. Takewaki (1997) used his incremental inverse problem approach for finding the optimal damper sizing to minimize the sum of the amplitudes of the transfer functions. Shukla and Datta (1999) dealt with the optimal insertion of viscoelastic dampers in frame structures. Takewaki (2000) formulated a new steepest direction search algorithm for finding the optimal damper positioning in shear-type structures subjected to critical excitation. Singh and Moreschi (2001) applied a gradient-based optimisation approach to determine the optimal distribution of supplemental damping in a 24-storey shear building to achieve a desired level of response reduction. Lopez-Garcia (2001) formulated a simplified sequential search algorithm which can be integrated into conventional design procedures for viscous dampers used by practicing engineers dealing with damper-added structures. Levy and Lavan (2006) faced the optimal damper insertion problem in framed structures defining a simple procedure to attain fully stressed behaviour of damper and the satisfaction of prescribed inter-storey drifts. In the latest decade, significant works regarding optimal damper configuration have also been published by many researchers (Singh and Moreschi 2002, Garcia and Soong 2002, Uetani *et al*. 2003, Lavan and Levy 2006a and 2006b, Liu *et al*. 2005, Aydin *et al*. 2007, Cimellaro 2007, Cimellaro and Retamales 2007, Fujita *et al* 2010a and 2010b, Takewaki 2009, Lavan and Levy 2010).

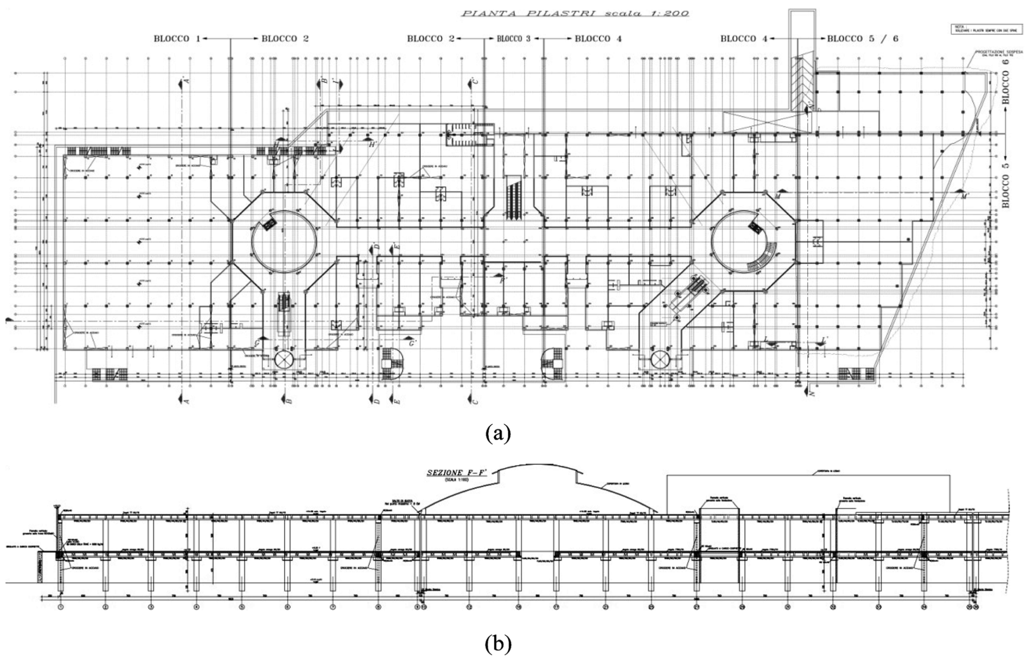
However, in almost all cases, valuable but sophisticated algorithms and/or efficient but complex procedures have been proposed which may hardly represent a direct and immediate help for the practitioner engineers. Thus, the issue of a simple guided procedure (other than the commonly adopted trial-and-error approach) capable of identifying the mechanical characteristics (c, α, k_{oil}) which allow to achieve target levels of seismic performances has been still not completely faced and solved. In this respect, only two contributions can be found in the scientific/technical literature.

First, Christopoulos and Filiatrault (2006) suggested a practical design procedure for estimating the damping constants of individual dampers on the basis of the knowledge of the inter-storey lateral stiffness and the introduction of a "generalised stiffness coefficient of a fictitiously braced structure". Second, a much simpler alternative procedure based upon the knowledge of the floor masses has been recently introduced by the authors (Silvestri *et al*. 2010), as originally developed for shear-type building structures.

In the light of the above framework, in this paper, a case-study is developed with reference to a precast reinforced concrete building structure (two-storey shopping mall recently built in northern Italy, Rimini), which aims at both (i) presenting an example of application of the procedure introduced in (Silvestri *et al*. 2010) to check its applicability to "pendular" systems and (ii) giving an account of the whole design process and the analyses developed for the executive project of the structure under consideration, with special care regarding the dimensioning of dampers and the assessment of their effectiveness in mitigating the seismic effects upon the precast structure. This article aims at representing a useful contribution to the practicing engineering community and at making the scientific community to reflect upon the importance of devoting specific efforts into developing the necessary transition between the research and its application, which is not a trivial process, but it has itself a scientific value.

2. The reference building structure

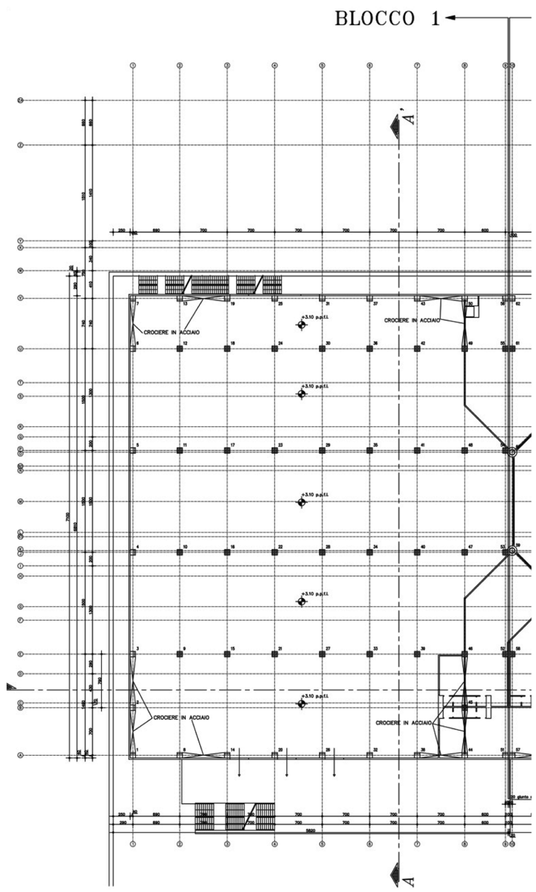
The case study is a two-storey precast reinforced concrete structure of a shopping mall characterised by a total surface of about 40,000 m^2 (two floors of about 20,000 m^2 each). From a



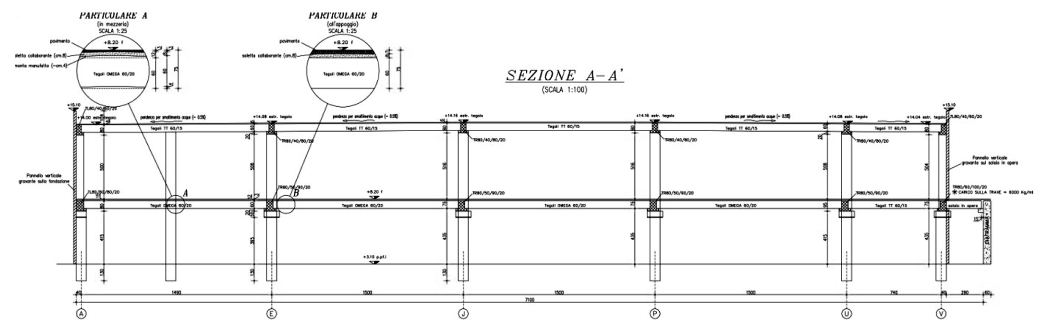
(a)

(b)

Fig. 2 (a) Plan-view (also indicating the subdivision in six building structures) and (b) longitudinal section of the whole shopping mall (technical drawings kindly provided by G.E.D. construction firm) structural point of view, the mall is divided in six different building structures separated by appropriately-sized seismic joints. Figs. 2(a) and 2(b) represent the plan-view and the longitudinal section of the whole shopping mall, respectively. This paper focuses on the description of the seismic design of Building 1 ("BLOCCO 1" in the technical drawings, which is located on the left-



(a)

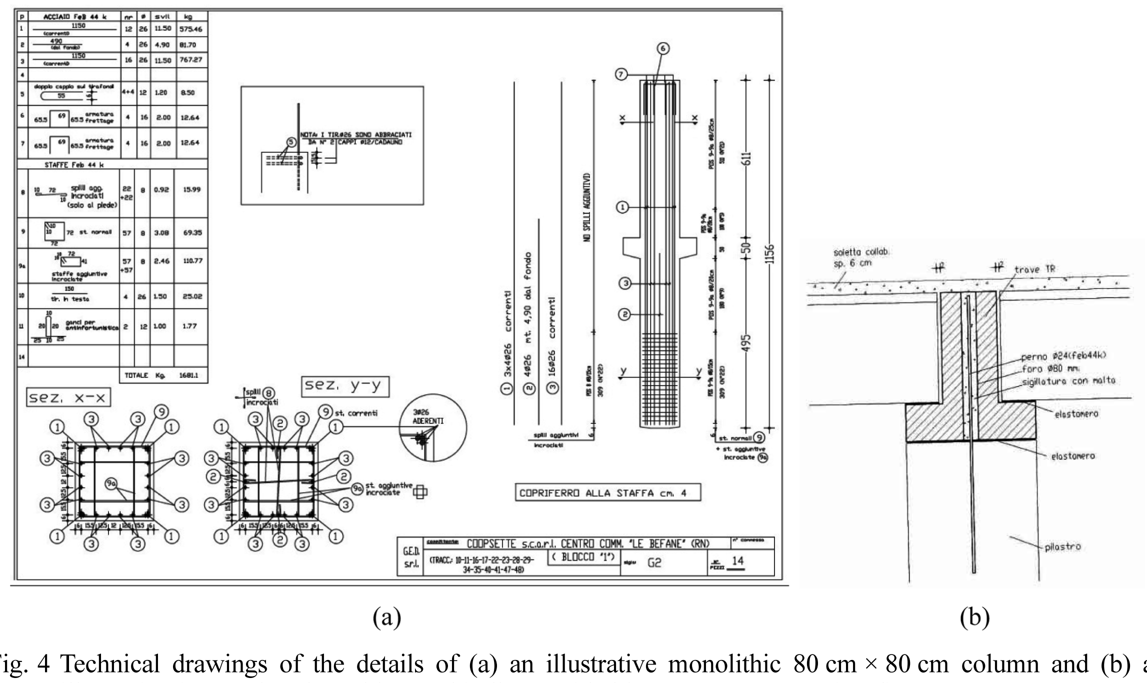


(b)

Fig. 3 (a) Plan-view and (b) section along the *Y*-direction of Building 1 (technical drawings kindly provided by G.E.D. construction firm) hand side of the plan represented in Fig. 2(a)). Figs. 3(a) and 3(b) represent the plan-view and the section along the *Y*-direction of Building 1, respectively.

Building 1 has a rectangular plan (of about 55 m × 67 m) and a maximum height of about 10.50 m (it is composed of ground floor at 0.00 m, first floor at 4.55 m and roof at 10.50 m). Overall, the total floor area of Building 1 is about 8000 m^2. The floors are built to bear live loads (in addition to self weights and dead loads) of 7.00 kN/m^2 (at the ground and the first floors) and of 1.30 kN/m^2 (at the roof floor), depending on the different use allocations. This gives a total mass of about $m_1 = 5900$ kN·s^2/m for the first storey and of about $m_2 = 3000$ kN·s^2/m for the roof storey. The maximum deformations of the floors, due to live loads only, turn out to be (in compliance with the regulation) lower than 1/1000 of the floor span. The columns and the beams are realized with precast reinforced-concrete elements (concrete of class C45/55 and steel bars of type B450C) which are dry-assembled without any casting "in situ". This structural typology is therefore characterised by not moment-resisting frames in both *X*- and *Y*-directions. The building has light outdoor steel fire staircases along the *X*-direction, which are independent from the main structure. Thus, they have been neglected in the following analyses.

In detail, the columns (Fig. 4(a)) are monolithic elements characterised, in general, by cross-section equal to 80 cm × 80 cm, from which appropriate supports (stocky cantilevers) for the beams stand out for a length of about 35 cm. Prestressed concrete beams (of inverted *T* shape, as represented in Fig. 4(b)) lean on these cantilevers and, in turn, support the floors which are realized with precast prestressed concrete Ω-shaped load-bearing tiles filled with a 6 cm thick cast-in-situ concrete slab. The connections between beams and slab and those between slab and Ω-shaped load-bearing tiles are realized using steel pins dimensioned in order to ensure effective and safe transmission of the earthquake-induced actions. All the connections between columns and beams are



(a) (b)

Fig. 4 Technical drawings of the details of (a) an illustrative monolithic 80 cm × 80 cm column and (b) an illustrative inverted *T* shaped beam (kindly provided by G.E.D. construction firm) realized by means of the insertion of appropriate steel pins (which are drowned in the beams and in the cantilevers standing out from the columns and filled with adequate high-resistance mixtures) capable of transmitting shear forces, but not bending moments, to the columns.

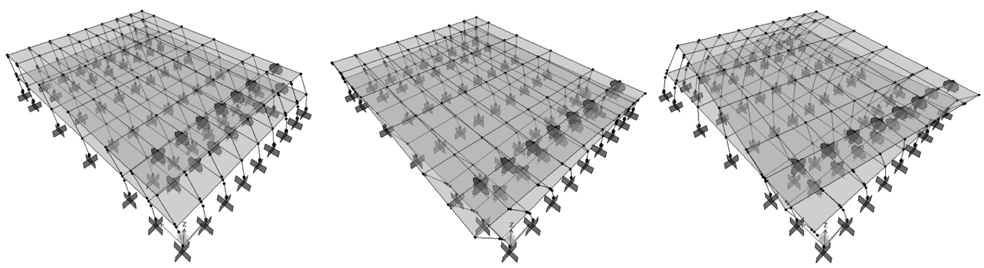
It is then clear that, with reference to the horizontal seismic actions, the columns act like cantilever elements (thus characterised by large flexibility). Consequently, the satisfaction of all structural requirements imposed by seismic codes (with special reference to the limitations upon the maximum structural displacements for frequent earthquakes, i.e. for Damage Limit State conditions) may prove to be a difficult task without the introduction of any earthquake-resistant bracing system.

3. The dynamic and seismic behaviour of the reference building structure (without any additional bracing or damping system)

3.1 Dynamic properties of the building structure

For the analysis of the earthquake-induced actions upon the structure, a three-dimensional finite element model has been developed. Columns and beams have been modelled with “beam” elements, floor slabs with “shell” elements and diagonal braces with “beam” elements in the case of rigid bracing systems and with appropriate “non-linear link” elements in the case of dissipative bracing systems. To reduce the computational time of the analyses, a number of 36 modes has been considered in the dynamic analyses; this is sufficient to activate more than 85% of the mass in both *X*- and *Y*-directions, as usually prescribed by seismic codes (specifically, Eurocode 8). Special attention has been devoted to the modelling of the connections between the shell elements (representing the Ω-shaped tiles + concrete slab) and the beams to impose that forces could not be transmitted in the model directly between the shell elements and the beam elements representing the columns (as it is the case of the actual building).

The reference structure is characterised by the following periods of vibration for the first three modes: $T_1 = 1.30$ s, $T_2 = 1.22$ s and $T_3 = 1.13$ s (the first and second modes are translational along the *Y*- and *X*-directions, the third mode is rotational). Figs. 5(a), 5(b) and 5(c) represent the deformed shapes of the first three modes. Notice that the slight difference between the periods of the first two translational modes is due to the perimeter pillars, which for architectural reasons are characterised by rectangular 80 cm × 60 cm (instead of square 80 cm × 80 cm) cross-section (see Fig. 7 for their plan localisation).



(a) (b) (c)

Fig. 5 Deformed shape of the first three modes of vibration for the reference structure: (a) mode 1, (b) mode 2, and (c) mode 3

3.2 The seismic inputs

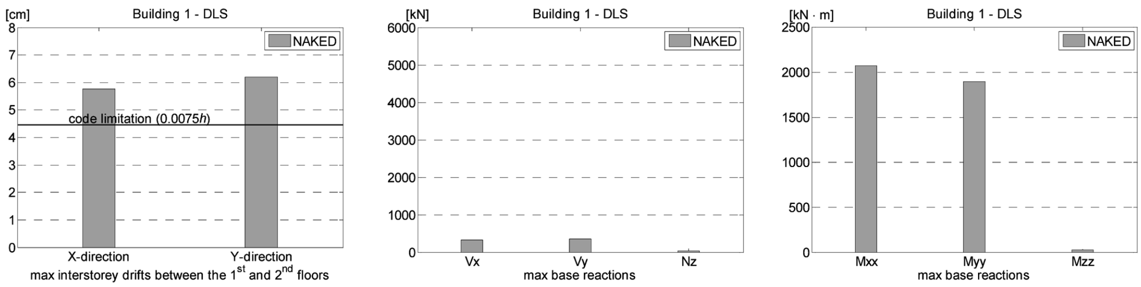
For the seismic analysis of the building structure, according to the design provisions of Eurocode 8, seven groups of accelerograms were created synthetically. Each group consisting of three different accelerograms to be considered in the numerical analysis as acting simultaneously along the *X*-, the *Y*- and the *Z*-directions. The synthetic accelerograms were used to simulate (by proper scaling) both the seismic action with an occurrence probability equal to 50% in 50 years, corresponding to the Damage Limit State (DLS), and the seismic action with an occurrence probability equal to 10% in 50 years, corresponding to the Ultimate Limit State (ULS). The synthetic accelerograms used are obtained using the "SIMQKE" software and match the elastic spectrum provided by Eurocode 8, so that no value of the mean 5% damping elastic spectrum - calculated from all time-histories - is less than 90% of the corresponding value of the 5% damping elastic response spectrum.

For the building at hand, the seismic inputs to be used for the DLS are characterised by a peak ground acceleration equal to: 0.10 g, while those to be used for the ULS are characterised by a peak ground acceleration equal to: 0.25 g. According to Eurocode 8, these actions were increased by 20% to account for the special use of the building (large crowd must be accounted for a shopping mall: importance class II) and the seismic design of the building structure (here developed by means of linear and non-linear time-history analyses) was developed with reference to the mean structural response to seven groups of earthquake inputs.

3.3 Seismic behavior of the building structure

Seismic analysis were performed for both the DLS and the ULS. According to Eurocode 8, in order to satisfy the requirements of the DLS it is necessary that the inter-storey drifts of the structure do not exceed specific limits. On the other hand, in order to satisfy the requirements for the ULS, the structure must show both specific capabilities in terms of strength as well as limitations in the maximum deformations in order to prevent pounding and considerable second order effects. The main issue in the design of the structure, as given in detail below, proved to be the satisfaction of the limitation of the maximum structural deformations for the DLS conditions.

Figs. 6(a), 6(b) and 6(c) give the maximum values (among the structural nodes) of the averages



(a) (b) (c)

Fig. 6 Maximum values (among the structural nodes) of the averages (over the seven groups of accelerograms) of (a) maximum inter-storey drifts, (b) shear and axial forces, (c) bending and torsional moments, for the reference building structure (DLS) (over the seven seismic inputs) of the inter-storey drifts and of the base joint reactions, as obtained under the DLS inputs for the so-called “NAKED solution”: original structural system composed of precast columns and beams described in section 2 without the addition of any bracing system.

The NAKED structure displays values of inter-storey drifts between the first and the second floors which exceed the maximum one allowed by Eurocode 8 (roughly 6 cm vs. a limit of $0.0075 \cdot h = 0.0075 \cdot 595 \cong 4.46$ cm for buildings having ductile non-structural elements). An increase in the column cross-section proved (in analyses not reported herein for sake of conciseness) to be inefficient for a sufficient reduction of the inter-storey drifts. For this reason, the introduction of earthquake-resistant (either stiff or dissipative) bracing systems was necessary, as described in the following section.

4. The structural solutions considered

Due to architectonical and distributive reasons, the necessary earthquake-resistant bracing systems cannot be spread over the whole building plan and could only be placed in the eight bays represented in Fig. 7. Insofar as it is consistent with architectonical requirements, the choice of the bracing system positions has been studied in such a way as to optimise their plan disposal in order to minimize the torsional effects on the structure.

In addition to the reference structure without any kind of added bracing or damping systems (referred to as the “NAKED” solution), three distinct structural solutions encompassing either stiff or dissipative bracing systems have been taken into consideration during the design process:

- Precast structure with steel Inter-Storey Bracing (hereafter referred to as the “ISB” solution);
- Precast structure with Fixed-Point Dampers placed so that they connect each floor to the ground (hereafter referred to as the “FPD” solution);
- Precast structure with Inter-Storey Dampers (hereafter referred to as the “ISD” solution).

It is worth pointing out that, in all analyses, for sake of conservativeness of the results, internal damping is neglected for the FPD and ISD solutions, while a 5% damping is applied to each mode

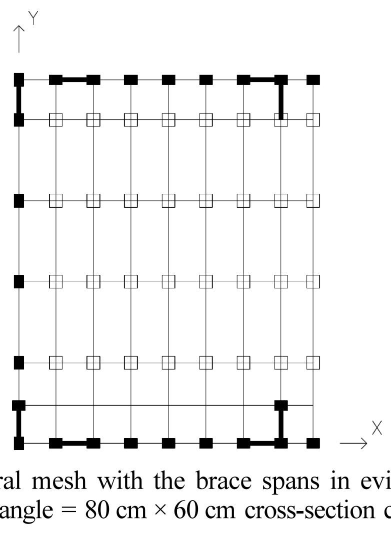


Fig. 7 Schematization of the structural mesh with the brace spans in evidence and with the indication of the size of the columns (black rectangle = 80 cm × 60 cm cross-section column, white square = 80 cm × 80 cm cross-section column)

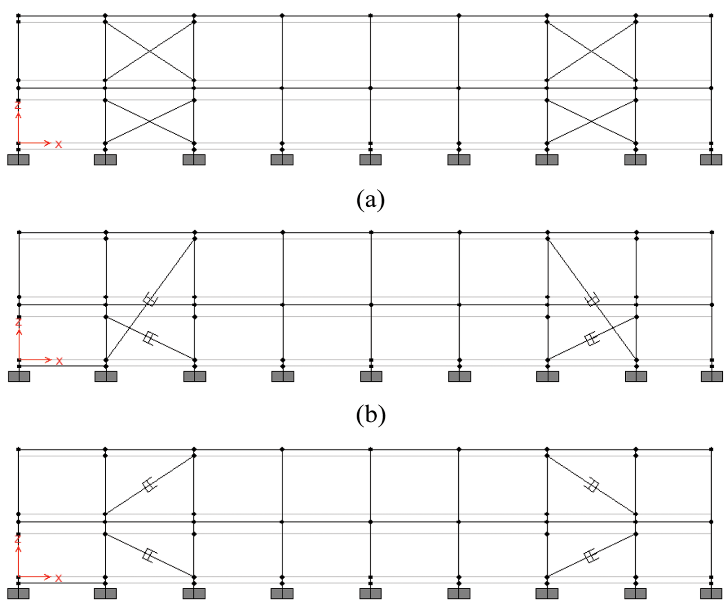
for the NAKED and ISB solutions.

The ISB solution encompasses steel diagonal stiff braces (following an "*X*" placement as per Figs. 8(a) and 9(a)) in all the eight bays indicated in Fig. 7. The bracing system at the first storey is made up using two UPN 220 profiles (coupled together). The bracing system at the second storey is made up using two UPN 260 profiles (coupled together). Indefinite elastic behaviour of these braces is assumed (section 11 will present another structural solution with non-linear behaviour of the braces). The periods of vibration of the first three modes are: $T_1 = 0.65$ s, $T_2 = 0.63$ s and $T_3 = 0.46$ s.

The FPD solution encompasses the insertion of linear viscous dampers (following the placement represented in Figs. 8(b) and 9(b)) in all the eight bays indicated in Fig. 7. Note that four dampers (two in each damped frame, as represented in Fig. 8(b)) connect each floor to the ground along the *X*-direction, and four dampers (two in each damped frame, as represented in Fig. 9(b)) connect each floor to the ground along the *Y*-direction. The lateral stiffness values of the structural elements required to transfer forces to damping devices to the base of the structure are dimensioned so that they could be considered as infinitely stiff.

The ISD solution encompasses the insertion of linear viscous dampers (following the placement represented in Figs. 8(c) and 9(c)) in all the eight bays indicated in Fig. 7. Note that four dampers (two in each damped frame, as represented in Fig. 8(c)) connect each floor to the adjacent one along the *X*-direction, and four dampers (two in each damped frame, as represented in Fig. 9(c)) connect each floor to the adjacent one along the *Y*-direction. The lateral stiffness values of the structural elements required to transfer forces to damping devices to the seismic force resisting system are dimensioned so that they could be considered as infinitely stiff.

The dimensioning of the viscous dampers of both the FPD and the ISD solutions is achieved using the procedure proposed by the authors in (Silvestri *et al*. 2010) and detailed in the following sections:



(a)

(b)

(c)

Fig. 8 Position of the braces in plan *XZ* for structural solutions: (a) ISB, (b) FPD and (c) ISD

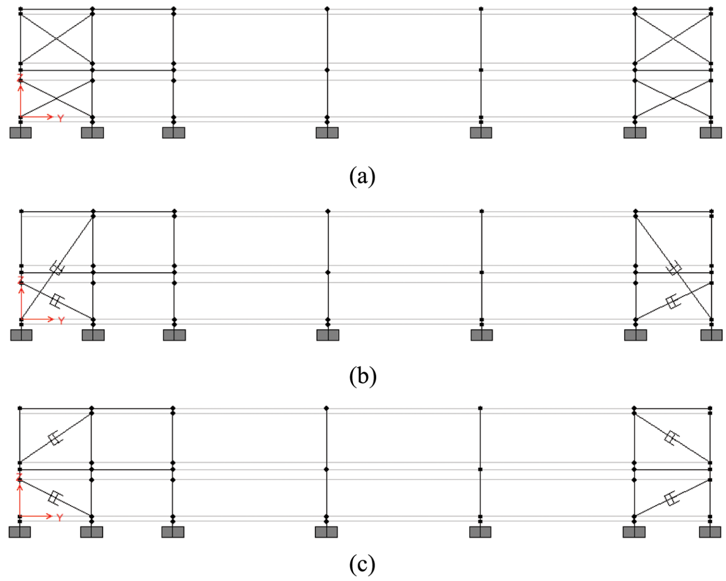


Fig. 9 Position of the braces in plan *YZ* for structural solutions: (a) ISB, (b) FPD and (c) ISD

- Section 5 presents a brief overview of the procedure for the identification of the mechanical characteristics of the viscous dampers.
- Sections 6, 7, 8, 9 and 10 provide the details of the whole procedure developed starting with reference to a target damping ratio identified on basis of the DLS requirements, which proved to govern the sizing of the structural elements resisting to the horizontal loads (columns).
- Section 11 provides the comparison of the results which can be achieved according to the final feasible structural solutions with reference to the ULS conditions, in order to identify the amount of reinforcement to be placed in the structural elements.

5. Overview of the procedure for the identification of the mechanical characteristics of the viscous dampers

The five-step procedure for the identification of the mechanical characteristics of the viscous dampers (Silvestri *et al.* 2010) is simply based on the knowledge of the floor masses and the fundamental period of vibration of the structure, relies upon useful relationships between the first modal damping ratio and the damping coefficients obtained for shear-type building structure schematization with equal mass and equal lateral stiffness at every story, and may be summarised as follows:

STEP 1: identification of the target damping ratio $\bar{\xi}$ of the structure on the basis of a chosen target level $\bar{\eta}$ of structural performances.

STEP 2: identification of the tentative characteristics of the linear viscous dampers for preliminary design ($c_L = \overline{c_L}$, $\alpha = 1.0$, $k_{oil} = \infty$), i.e. first dimensioning of the linear damping coefficients.

STEP 3: development of a series of preliminary time-history analyses of the building structure equipped with viscous dampers identified in Step 2. This step allows to: (i) calibrate the linear damping coefficients of the dampers to be added in the structure in order to achieve the desired level of actions (axial forces, shear forces, bending moments, etc.) on the structural members of the building; and (ii) identify the range of "working" velocities for the linear added viscous dampers.

STEP 4: identification of the characteristics of the "equivalent" non-linear viscous dampers ($c_{NL} = \overline{c_{NL}}$, $\alpha = \overline{\alpha}$, $k_{oil} = \overline{k}_{oil}$), i.e. identification of a system of manufactured viscous dampers capable of providing the structure with actions (on the structural members) comparable to those obtained in Step 3 using the linear viscous dampers identified in Step 2.

STEP 5: development of a series of final time-history analyses of the building structure equipped with the viscous dampers identified in Step 4. This last step being necessary in order to verify the effectiveness of Step 4 and obtain actions both on the structural members and on the dampers to be used for the final design specifications.

For more details and for all the notations which will be used hereafter, the interested reader is referred to the work by Silvestri *et al*. (2010).

6. STEP 1: Identification of the target damping ratio of the structure

From the DLS results obtained for the reference structure and reported in section 3.3, the minimum reduction factor (necessary for the satisfaction of the limitation of the maximum structural deformations) can be estimated as

$$\eta_{min} = \frac{4.46}{6} \cong 0.75 \tag{1}$$

Taking into consideration the optimal values for ξ recalled in (Silvestri *et al*. 2010), it has been decided to exceed this minimum value of damping performances and to select a target damping ratio $\overline{\xi}$ of about 0.35, which should lead, according to many established formulations available in scientific literature (Cardone *et al*. 2007, Bommer *et al*. 2000, Tolis and Faccioli 1999, Italian SSN 1998, Priestley 2003, Kawashima and Aizawa 1986), to a target reduction factor of about $\overline{\eta} = 0.50$ in the system response. This also accounting for the strong approximations in the simplified procedure here adopted, i.e. the structure at hand is far away from the equal-mass and equal-stiffness shear-type model considered for the development of the simplified procedure described in (Silvestri *et al*. 2010).

7. STEP 2: Identification of the preliminary characteristics of the viscous dampers (linear dampers)

The specific characteristics of the building structure at hand (most of the pillars have a square cross-section) leads to a dynamic behaviour which is similar along both *X*- and *Y*-directions, as given in section 3.1. For this reason, the two-dimensional schematisation required in the procedure for the identification of the characteristics of the viscous dampers will be here developed with reference to the *X*-direction only (with results thus obtained extended also for the *Y*-direction).

7.1 Fixed-point dampers

With reference to the exact procedure described in section 5.1 of the paper by Silvestri *et al*.

(2010) (note that, for Fixed-Point Dampers, the development of the exact procedure represents an easy task, given that it is sufficient to know the floor masses m_i, the fundamental circular frequency of the system ω_1 along the X-direction and the target damping ratio $\bar{\xi}$)

$$\omega_1 = \frac{2\pi}{T_1} = \frac{2\pi}{1.22\mathrm{s}} = 5.15\ \frac{\mathrm{rad}}{\mathrm{s}} \tag{2}$$

$$\bar{\xi} = 0.35 \tag{3}$$

With reference to $n = 4$ dampers per floor, Eq. (12) of (Silvestri *et al.* 2010) gives

$$c_1 = 2 \cdot \bar{\xi} \cdot \omega_1 \cdot \frac{m_1}{n} = 2 \cdot 0.35 \cdot 5.15 \cdot \frac{5900}{4} = 5317 \frac{\mathrm{kN \cdot s}}{\mathrm{m}} \tag{4}$$

$$c_2 = 2 \cdot \bar{\xi} \cdot \omega_1 \cdot \frac{m_2}{n} = 2 \cdot 0.35 \cdot 5.15 \cdot \frac{3000}{4} = 2704 \frac{\mathrm{kN \cdot s}}{\mathrm{m}} \tag{5}$$

where c_1 and c_2 indicate the damping coefficient of the dampers to be placed at the first and at the second storey, respectively. To account for the actual inclination of the diagonal dampers (characterised by angles φ with respect to the horizontal axis), the "horizontal" damping coefficients should be corrected as follows

$$c_{1,inclined} = 5317 \frac{\mathrm{kN \cdot s}}{\mathrm{m}} \cdot \left(\frac{1}{\cos^2 27^{\mathrm{o}}} \right) = 6697 \frac{\mathrm{kN \cdot s}}{\mathrm{m}} \tag{6}$$

$$c_{2,inclined} = 2704 \frac{\mathrm{kN \cdot s}}{\mathrm{m}} \cdot \left(\frac{1}{\cos^2 55^{\mathrm{o}}} \right) = 8219 \frac{\mathrm{kN \cdot s}}{\mathrm{m}} \tag{7}$$

7.2 Inter-storey dampers

According to the simplified procedure described in section 5.4 of the paper by Silvestri *et al.* (2010) (note that for Inter-Storey Dampers, the development of the exact procedure may prove to be cumbersome)

$$m_{tot} = 5900 + 3000 = 8900 \frac{\mathrm{kN \cdot s^2}}{\mathrm{m}} \tag{8}$$

$$c_{total} = \bar{\xi} \cdot \omega_1 \cdot m_{tot} \cdot N(N+1) = 0.35 \cdot 5.15 \cdot 8900 \cdot 2 \cdot 3 = 96254 \frac{\mathrm{kN \cdot s}}{\mathrm{m}} \tag{9}$$

With reference to $n = 4$ dampers per floor, Eq. (25) of (Silvestri *et al.* 2010) gives

$$c_1 = c_2 = \bar{\xi} \cdot \omega_1 \cdot m_{tot} \cdot \left(\frac{N+1}{n} \right) = 0.35 \cdot 5.15 \cdot 8900 \cdot \frac{3}{4} = 12032 \frac{\mathrm{kN \cdot s}}{\mathrm{m}} \tag{10}$$

To account for the actual inclination of the diagonal dampers (characterised by angles φ with respect to the horizontal axis), the "horizontal" damping coefficients should be corrected as follows

$$c_{1,inclined} = 12032 \frac{\mathrm{kN \cdot s}}{\mathrm{m}} \cdot \left(\frac{1}{\cos^2 27^{\mathrm{o}}} \right) = 15156 \frac{\mathrm{kN \cdot s}}{\mathrm{m}} \tag{11}$$

$$c_{2,inclined} = 12032\frac{\mathrm{kN}\cdot\mathrm{s}}{\mathrm{m}}\cdot\left(\frac{1}{\cos^2 34^{\circ}}\right) = 17506\frac{\mathrm{kN}\cdot\mathrm{s}}{\mathrm{m}} \tag{12}$$

Even though the dampers inclination angles and thus the resulting theoretical "horizontal" damping coefficients are slightly different, the values of the damping coefficients were taken equal in the following calculations ($c_{1,inclined} = c_{2,inclined} = 15156\frac{\mathrm{kN}\cdot\mathrm{s}}{\mathrm{m}}$) for sake of economies of scale (uniformity leads to costs saving when asking a manufacturer for a quote). Note that this is a case in which, to accommodate specific practical issues, slight divergences from the theoretical framework are introduced. In this way, this paper aims also at providing insight on the effects on the final structural response of possible divergences between the mathematical and the practical world.

8. STEP 3: Preliminary time-history analyses

Once the tentative characteristics of the linear viscous dampers are identified (the damping coefficients have just been obtained in the previous section, while the axial stiffness may be assumed, to a first approximation, as $k_{oil} = \infty$), preliminary time-history dynamic analyses are performed using, as base dynamic inputs, the seven groups of acceleration time-histories, for both the DLS and the ULS conditions, described in section 3.2.

8.1 Results obtained: the seismic response of the structure

The results, synthetically represented in Figs. 10 and 11, are given in terms of maximum values (among the structural nodes or elements) of the averages (over the seven seismic inputs) of some significant response parameters (inter-storey drifts, forces in the diagonal braces, floor accelerations, base joint reactions) obtained for DLS. In order to have a complete picture of the outcome of the numerical simulations, the reference results of the NAKED solution, already discussed in section 3.3, are still reported in these figures.

For the ISB solution it can be observed that, for DLS, the values of the inter-storey drift between the first and the second storeys are considerably reduced (to about 2.3 cm in both *X*- and *Y*-directions) so that they satisfy the limits imposed by the code. However, this is reached at the expense of large floor accelerations and actions in the steel diagonal braces. The accelerations of the 2nd floor reach values of about 0.4 g, with an increase of +33% with respect to the corresponding values of the NAKED solution (about 0.3 g). The axial forces in the UPN elements reach values up to about $F_{max} \cong 1250$ kN on the 1st storey and $F_{max} \cong 2000$ kN on the 2nd storey. This leads to very large actions to be taken by the foundation ($N_z \cong 5000$ kN). As a matter of fact, this problem makes the ISB solution inapplicable.

For the FPD solution it can be observed that, for DLS, the inter-storey drift values are considerably reduced (to about 2.3 cm in the *X*-direction and 2.4 cm in the *Y*-direction) so that they satisfy the limits imposed by the code. Note that these maximum values of inter-storey drift (between the first and the second storeys) are similar to those obtained for the ISB solution. Nonetheless, this reduction is obtained maintaining the floor accelerations and the actions in the diagonal bracing system to relatively small values ($a_{max} \cong 0.3$ g and $F_{max} \cong 500$ kN on the 1st storey,

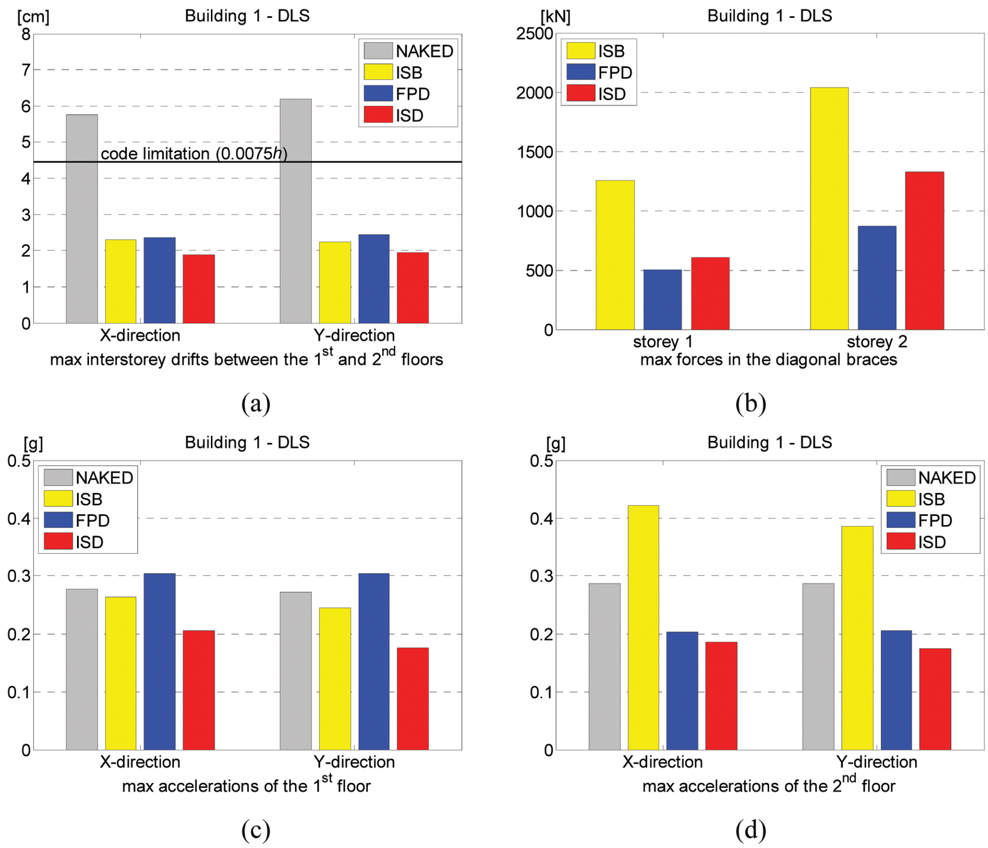


Fig. 10 Maximum values (among the structural nodes or elements) of the averages (over the seven groups of accelerograms) of the (a) maximum inter-storey drifts, (b) maximum forces in the diagonal braces, (c) maximum accelerations of the 1st floor, and (d) maximum accelerations of the 2nd floor for the structural solutions considered (DLS)

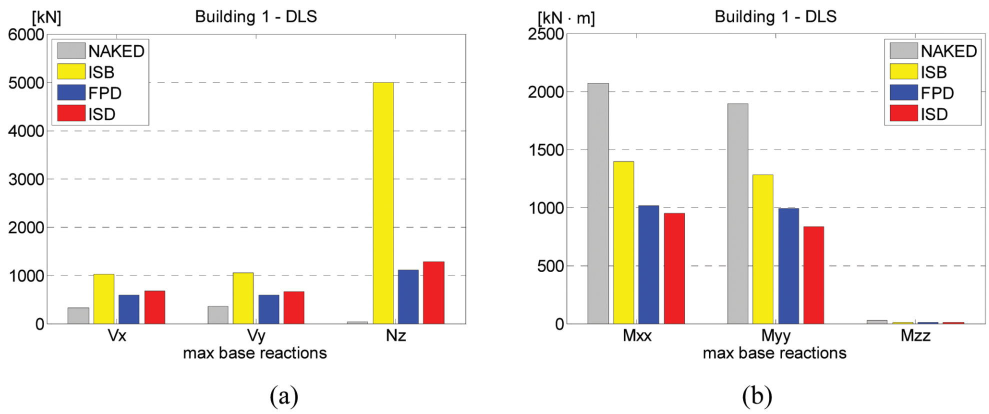


Fig. 11 Maximum values (among the structural nodes or elements) of the averages (over the seven groups of accelerograms) of the maximum base reactions for the structural solutions considered (DLS): (a) shear and axial forces, (b) bending and torsional moments

$a_{max} \cong 0.2$ g and $F_{max} \cong 850$ kN on the 2nd storey). This induces vertical actions upon the foundations ($N_z \cong 1100$ kN) which are considerably smaller (about 1/5) than those obtained for the ISB solution. Also, the bending moments at the basis of the columns ($M_X \cong M_Y \cong 1000$ kN·m) are smaller (of about $-25\% \div -30\%$) than those obtained for the ISB solution.

As far as the ISD solution is concerned, it is observed that its structural behaviour under seismic input is quite similar to that observed for the FPD solution (with the ISD solution being slightly more effective in reducing the structural deformations, floor accelerations and base bending moments, at the price of larger damper forces and axial forces upon the foundations).

In summary, the advantages deriving from the insertion of viscous dampers (either following the FPD or the ISD solutions) reside in (i) substantial reductions in deformations (and actions) of the structures (with respect to the NAKED structure), and (ii) reduced actions induced in the bracing systems and in the foundations (with respect to the ISB solution).

8.2 Results obtained: the maximum relative velocities between each end of the dampers

The preliminary time-history analyses performed using a linear constitutive model for the dampers allow also to obtain the maximum damper velocities which are fundamental for the final technical specifications of the manufactured non-linear dampers. Table 1 provides the maximum velocities (over all damping devices of a given size), for ULS, developed by the dampers of the FPD and the ISD solutions, as subjected to the seven design seismic inputs considered.

9. STEP 4: Identification of the characteristics of the "equivalent" non-linear viscous dampers

This section gives the details of how the final technical specifications of the manufactured non-linear viscous dampers are obtained.

For sake of conciseness, let's here take into account only the dampers connecting the roof to the ground (c_2) of the FPD solution. For such devices, the maximum "linear" damper velocity is $v_{max} = 0.320$ m/s. Notice that, for sake of conservativeness of the expected results, the maximum value between all maximum values over the seven design seismic records (as given by Table 1) has been taken into account (even though the code allows to design the building for the average response to the seven inputs). Considering $\overline{\alpha} = 0.3$ and $\chi = 0.8$, Eq. (28) of (Silvestri *et al*. 2010) gives

Table 1 Maximum damper velocities (ULS) for the FPD and the ISD solutions

Structural solution	Storey	Maximum damper velocities (ULS) [m/s]						
		Seismic input						
		n.1	*n*.2	*n*.3	*n*.4	*n*.5	*n*.6	*n*.7
FPD	1	0.217	0.203	0.227	0.212	0.202	0.200	0.223
FPD	2	0.315	0.320	0.284	0.272	0.284	0.308	0.298
ISD	1	0.122	0.128	0.110	0.101	0.104	0.119	0.114
ISD	2	0.259	0.245	0.246	0.235	0.235	0.229	0.226

$$\overline{c_{2,NL}} \cong \overline{c_{2,L}} \cdot (0.8 \cdot v_{max})^{1-\overline{\alpha}} = 8219 \cdot (0.8 \cdot 0.320)^{1-0.3} = 3167 \frac{\mathrm{kN} \cdot \mathrm{s}^{0.3}}{\mathrm{m}^{0.3}} \tag{13}$$

As illustrative example, Fig. 12 shows the “linear” and “non-linear” force-velocity relationships for the dampers connecting the roof to the ground (c_2) of the FPD solution.

As far as the minimum damper axial stiffness is concerned, for the dampers connecting the roof to the ground (c_2) of the FPD solution, Eq. (29) of (Silvestri *et al.* 2010) gives

$$\bar{k}_{oil,2} \geq k_{oil,min,2} = 10 \cdot \overline{c_{2,L}} \cdot \omega_1 = 10 \cdot 8219 \cdot 5.15 = 4.23 \cdot 10^5 \frac{\mathrm{kN}}{\mathrm{m}} \tag{14}$$

In the numerical dynamic analyses (and in the technical specifications for the manufacturer) it has been used the value: $\bar{k}_{oil,2} = 5 \cdot 10^5 \frac{\mathrm{kN}}{\mathrm{m}}$.

Similar considerations can be developed for the other dampers of the FPD and ISD solutions. Table 2 provides the final design values of the “non-linear” mechanical characteristics of all the manufactured viscous dampers of both the FPD and ISD structural solutions.

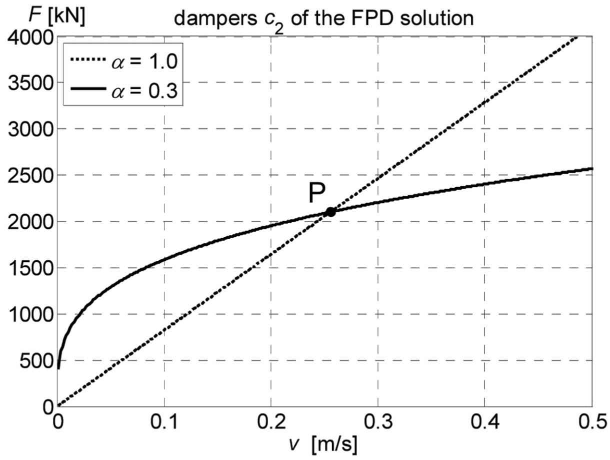


Fig. 12 “Linear” and “non-linear” force-velocity relationships for the dampers connecting the roof to the ground (c_2) of the FPD solution

Table 2 Final design values of the “non-linear” mechanical characteristics of all the manufactured viscous dampers of both the FPD and ISD structural solutions

Structural solution	Storey	v_{max} [m/s]	c_L [kNs/m]	α	c_{NL} [kN(s/m)$^{0.3}$]	$k_{oil,min}$ [kN/m]	$\bar{k}_{oil}$ [kN/m]
FPD	1	0.227	6697	0.3	2029	3.45E+05	4.00E+05
FPD	2	0.320	8219	0.3	3167	4.23E+05	5.00E+05
ISD	1	0.128	15156	0.3	3075	7.81E+05	8.00E+05
ISD	2	0.259	15156	0.3	5036	7.81E+05	8.00E+05

10. STEP 5: Final time-history analyses

Finally, numerical time-history analyses are performed using the non-linear constitutive model of the dampers identified in the previous section. The concrete structure is modelled as a linear elastic given that it is assumed to behave elastically for both DLS and ULS states ("response reduction factor" or "behaviour factor" assumed equal to 1.00). The seven groups of synthetic acceleration time-histories have been used as base input.

10.1 Results obtained for DLS

Fig. 13 compares the maximum values (among the structural nodes or elements) of the averages (over the seven groups of accelerograms) of the maximum inter-storey drifts for DLS, as obtained for the FPD and the ISD solutions which make use of the linear constitutive model for the dampers (hereafter referred to as FPD-L and ISD-L solutions) and the FPD and the ISD solutions which make use of the non-linear constitutive model for the dampers (hereafter referred to as FPD-NL and ISD-NL solutions).

It can be seen that, for both the FPD and the ISD cases, the responses of the systems which make use of the non-linear constitutive model for the dampers (i) are similar to those of the corresponding systems which make use of the linear constitutive model (thus showing the effectiveness of the equivalence criteria imposed in Step 4), and (ii) satisfy the inter-storey drift limitations imposed by Eurocode 8 (thus proving the effectiveness of the overall procedure for the dimensioning of the manufactured viscous dampers).

10.2 Results obtained for ULS

Fig. 14 compares the maximum values (among the structural nodes or elements) of the averages (over the seven groups of accelerograms) of the base reactions for ULS, as obtained for the FPD-L,

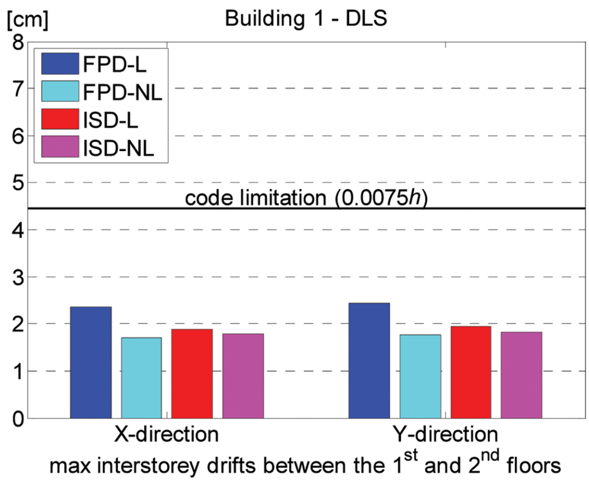
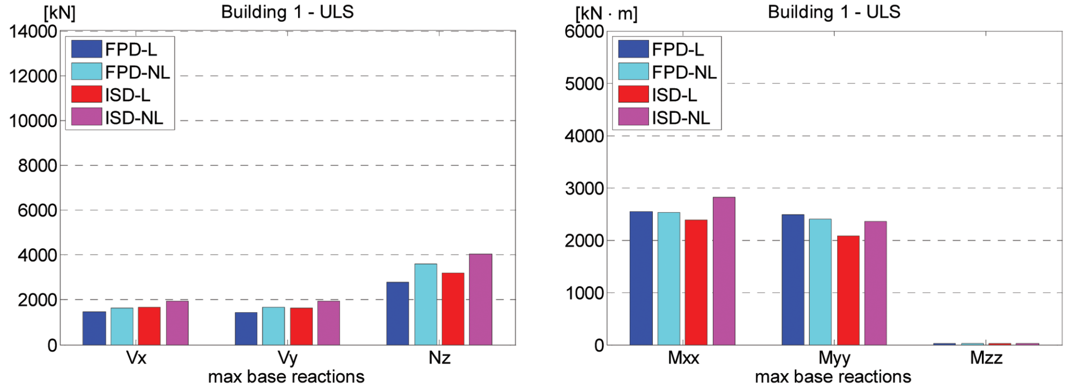


Fig. 13 Maximum values (among the structural nodes or elements) of the averages (over the seven groups of accelerograms) of the maximum inter-storey drifts as obtained for the FPD, ISD, FPD-NL and ISD-NL structural solutions (DLS)



(a) (b)

Fig. 14 Maximum values (among the structural nodes or elements) of the averages (over the seven groups of accelerograms) of the maximum base reactions for the FPD, ISD, FPD-NL and ISD-NL structural solutions (ULS): (a) shear and axial forces, (b) bending and torsional moments

the FPD-NL, the ISD-L, and the ISD-NL solutions. Again, as it is the case of the structural deformations for the DLS input, the responses of the systems which make use of the non-linear constitutive model for the dampers are similar to those of the corresponding systems which make use of the linear constitutive model.

11. Comparison of the performances offered by the ISB, FPD-NL and ISD-NL solutions and discussion

This section compares the performances offered by the ISB, FPD-NL and ISD-NL solutions only (the NAKED solution being inapplicable for DLS considerations, whilst the FPD and ISD solutions being preliminary configurations just used for the identification of the FPD-NL and ISD-NL solutions), in order to directly highlight and discuss the beneficial effects of the introduction of viscous dampers into a precast r.c. structure with respect to a traditional solution composed of stiff braces.

Another structural solution is here introduced and compared which encompasses the use of Buckling Restrained Braces instead of stiff braces (following the same "*X*" placement as per Figs. 8(a) and 9(a) in all the eight bays indicated in Fig. 7 of the ISB solution). This solution (referred to as the "BRB" solution) is made up of braces which have: (i) the same initial stiffness of the two UPN 220 profiles on the 1st storey and the same initial stiffness of the two UPN 260 profiles on the 2nd storey, (ii) yielding forces equal to 1.2 times the DLS forces (i.e. 1500 kN on the 1st storey and 2400 kN on the 2nd storey), (iii) post-yielding stiffnesses equal to 3% of the initial ones, and (iv) Takeda hysteresis type. This constitutive model leads to the same elastic characteristics, same dynamic behaviour, and same DLS seismic response of the ISB solution, but to a different ULS seismic response.

This comparison is carried out with reference to the ULS conditions only, using as base input the seven groups of synthetic acceleration time-histories compatible with the ULS response spectrum. The concrete structure is modelled as linear elastic given that it is assumed to behave elastically also for ULS ("response reduction factor" or "behaviour factor" assumed equal to 1.00).

Fig. 15(a) compares the maximum values (among the structural nodes) of the averages (over the seven groups of accelerograms) of the maximum inter-storey drifts for ULS. It can be seen that, for all cases, the maximum inter-storey drifts are of the same order of magnitude (about 5.7 cm for the ISB, about 4.3 cm for the BRB, about 5.1 cm for the FPD-NL, and about 4.8 cm for the ISD-NL) and substantially equal along the *X*- and the *Y*-directions.

Fig. 15(b) compares the maximum values (among the structural elements) of the averages (over the seven groups of accelerograms) of the maximum forces in the diagonal braces for ULS. It can be seen that the largest forces are provided by the ISB solution (3100 kN at 1st storey and 5100 kN at 2nd storey). This is mainly due to the effect of period reduction due to the stiffness added by the rigid braces. On the contrary, the BRB, the FPD-NL and the ISD-NL solutions provide substantially lower values of the bracing forces. The forces of the BRB solution are clearly limited to values slightly larger than the yielding values due to the specific non-linear model adopted. It is worth pointing out here that there is a double number of BRB braces with respect to the number of

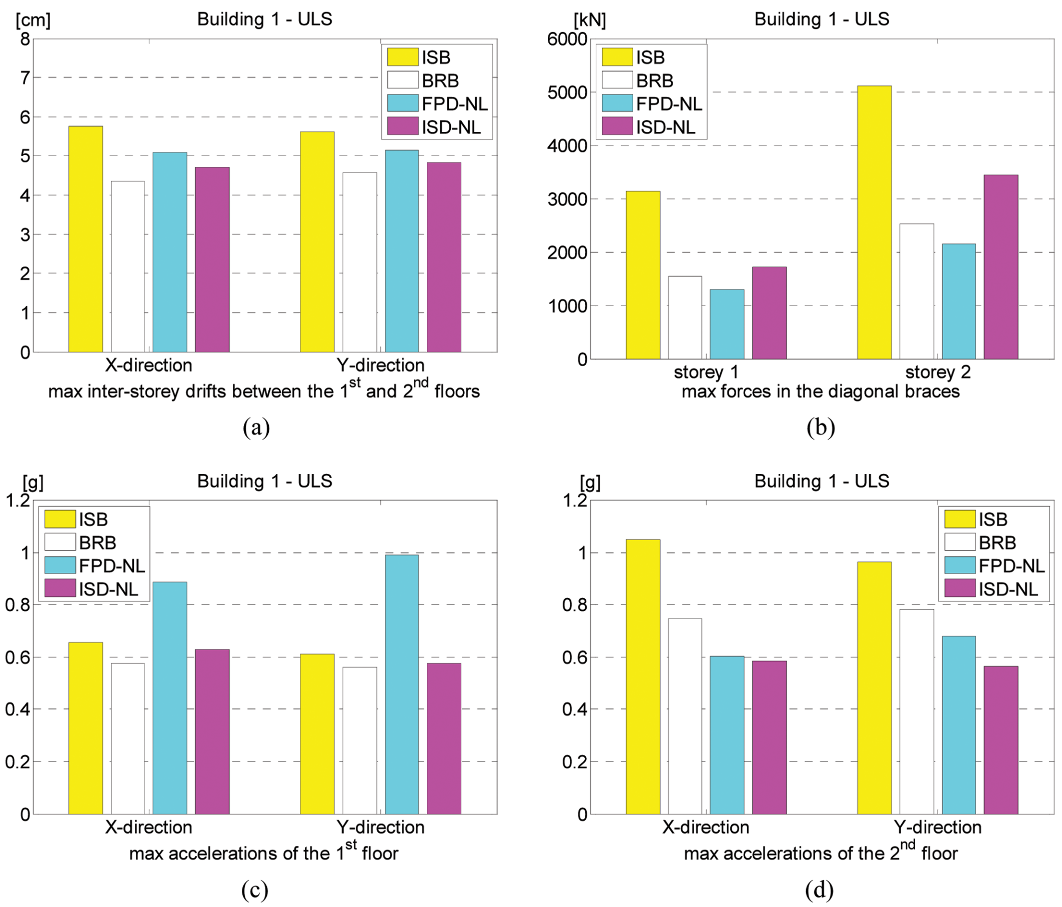
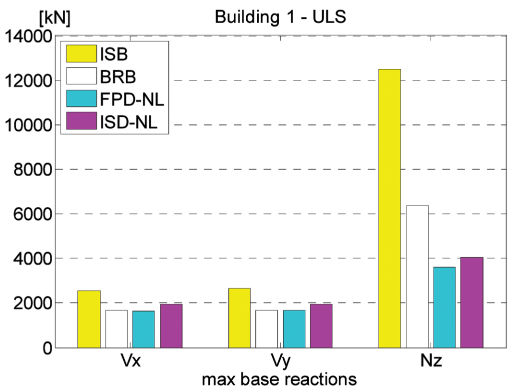


Fig. 15 Maximum values (among the structural nodes or elements) of the averages (over the seven groups of accelerograms) of the (a) maximum inter-storey drifts, (b) maximum forces in the diagonal braces, (c) maximum accelerations of the 1st floor, and (d) maximum accelerations of the 2nd floor for the structural solutions considered (ULS)

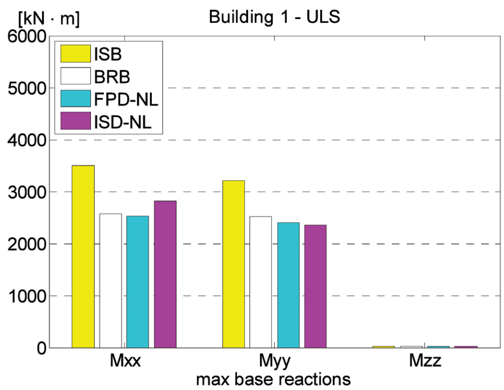
dampers in the FPD-NL and ISD-NL. The ISD-NL solution requires forces which are roughly 1.3 (at 1st storey) and 1.6 (at 2nd storey) times the corresponding ones of the FPD-NL solution. This last result should be read together with the previous results in terms of inter-storey drifts: the ISD-NL is slightly more effective in terms of deformations at the expense of larger forces in the dissipative braces. It is clear that this result applies to the specific sizing identified in section 9 and that further calibration of damping coefficients may lead to slightly different responses. Anyway, a wide amount of results reported in previous research works (Silvestri *et al*. 2003, Trombetti and Silvestri 2004, 2006, 2007, Silvestri and Trombetti 2007) allows the authors to state that, in general and especially for shear-type frame building models, Fixed-Point Dampers seem to be characterised by intrinsic better performances with respect to Inter-Storey Dampers.

Figs. 15(c) and 15(d) compare the maximum values (among the structural nodes) of the averages (over the seven groups of accelerograms) of the maximum accelerations of the 1st and 2nd floor, respectively, for ULS. It can be seen that the maximum accelerations of the 1st storey are of the same order of magnitude (about 0.6 g) except the case of the FPD-NL (about 0.9 g), which is likely due to the specific sizing detailed in section 7.1 (further calibration of damping coefficients may lead to slightly different responses). The maximum accelerations of the 2nd storey are about 1.0 g for the ISB, about 0.8 g for the BRB, about 0.6 g for both the FPD-NL and the ISD-NL, and substantially equal along the *X*- and the *Y*-directions.

Figs. 16(a) and 16(b) compare the maximum values (among the structural nodes) of the averages (over the seven groups of accelerograms) of the reactions at the base joints (shear forces, axial forces and bending moments) to which the bracing systems are attached to. It can be seen that, for all cases, the maximum bending moments at the base of the columns are of the same order of magnitude, with the ISB solution leading to a roughly +30% larger action (about 3350 kNm for the ISB, about 2500 kNm for the BRB, about 2500 kNm for the FPD-NL, and about 2600 kNm for the ISD-NL) and practically equal along the *X*- and the *Y*-directions. This trend is substantially and rightly in accordance with the trend found for the inter-storey drifts. Similar considerations may be done also for the shear actions at the base of the columns. On the contrary, tremendous differences may be noted in the axial forces: 12500 kN for the ISB solution, 6400 kN for the BRB solution,



(a)



(b)

Fig. 16 Maximum values (among the structural nodes or elements) of the averages (over the seven groups of accelerograms) of the maximum base reactions for the structural solutions considered (ULS): (a) shear and axial forces, (b) bending and torsional moments against 3600 kN and 4000 kN for the FPD-NL and ISD-NL, respectively. Note that, despite the forces in the braces are similar (Fig. 15(b)), the axial force base reaction in the BRB is substantially larger with respect to the corresponding ones in the FPD-NL and ISD-NL, due to the double number of BRB braces with respect to the number of dampers.

Provided that the whole structural system is composed of: (i) elevation made up of concrete structural elements, (ii) foundation system, and (iii) additional bracing system, inspection of the above results indicate how the "traditional" stiff bracing system, the use of buckling restrained braces and the "innovative" systems which make use of added viscous dampers lead to similar actions upon the concrete structural elements. The advantages of the damper solutions are clear once the axial forces (traction/compression actions) at foundation level are considered. The results of the numerical time-history simulations show that use of steel stiff bracing systems and buckling restrained braces are capable of satisfying the deformability requirements at the expense of large forces upon the foundations. On the other hand, the insertion of viscous dampers allows, at the same time, to reduce the drifts of the structure and to keep the forces upon the foundations (and also in the bracing system) at acceptable levels. This is also due to the fact that viscous dampers are frequency-activated devices, so that the forces generated by linear viscous dampers in a structure are "out-of-phase" with the forces generated by the structural system. For these reasons, the damper solutions lead to a great saving in foundation costs and thus to a considerable saving in total costs. Furthermore, ductility resources, which are in any case available in the r.c. structural elements, may be also accounted for (e.g. in terms of a given force reduction factor) in addition to the viscous dissipative proprieties of the added fluid dampers. In this respect, it is worth pointing out that, to the knowledge of the authors, in scientific literature no methodology is available yet for the simultaneous use of hysteretic (excursion in the plastic field of the structural elements) and viscous (added dissipative devices) damping for the seismic protection of building structures. A preliminary work has been recently developed by Drusiani (2010) under the supervision of the authors. From the viewpoint of the costs of the additional bracing systems, it should be remembered that the forces are similar (for the BRB, FPD-NL and ISD-NL cases), but the number of devices is different (double number of BRB braces with respect to the number of dampers). To sum up, it can be therefore stated that the structure designed using added viscous dampers provides superior performances with respect to any other traditionally designed structure, in that it is capable of withstanding either the same seismic excitation by being subjected to minor damages, or heavier seismic excitations by being subjected to the same damages.

For sake of completeness of information, Tables 3 and 4 show the maximum piston-strokes and the maximum damper forces, respectively, (over all dissipative devices of a given size), as obtained using the ULS seven groups of synthetic acceleration time-histories as base input, for all the dissipative devices of both the FPD-NL and the ISD-NL solutions. Note that the maximum piston-strokes, as well as the damper forces, display relatively small dispersions with respect to the earthquake records (e.g. coefficients of variation in the range of $0.05 \div 0.12$ for the piston-strokes and in the range of $0.02 \div 0.04$ for the damper forces). This contributes to the robustness of the structural design which makes use of viscous dampers.

The results, in terms of actions on all structural members (columns, foundations and dampers), obtained in these analyses can be readily used for the final executive design of the building structure (i.e. final identification of the appropriate amount of reinforcement). As illustrative example, in the case of the ISD-NL solution, the bending moment value of 2600 kNm may be used for the design of the reinforcement at the base of the typical column of the building.

Table 3 Maximum piston-strokes (ULS) for the FPD-NL and the ISD-NL solutions

Structural solution	Storey	Maximum piston-strokes (ULS) [cm]						
		Seismic input						
		n.1	*n*.2	*n*.3	*n*.4	*n*.5	*n*.6	*n*.7
FPD-NL	1	1.51	1.30	1.88	1.49	1.54	1.37	1.47
FPD-NL	2	4.57	4.21	3.79	3.36	4.19	4.29	3.98
ISD-NL	1	1.28	1.34	1.43	1.31	1.37	1.47	1.40
ISD-NL	2	3.33	2.98	2.98	2.77	3.36	3.56	3.40

Table 4 Maximum damper forces (ULS) for the FPD-NL and the ISD-NL solutions

Structural solution	Storey	Maximum damper forces (ULS) [kN]						
		Seismic input						
		n.1	*n*.2	*n*.3	*n*.4	*n*.5	*n*.6	*n*.7
FPD-NL	1	1290	1342	1357	1238	1314	1393	1312
FPD-NL	2	2148	2138	2265	2227	2189	2117	2221
ISD-NL	1	1741	1864	1795	1692	1763	1774	1736
ISD-NL	2	3447	3477	3639	3541	3498	3466	3507

Provided that the column presents a 80 cm x 80 cm cross-section, a concrete cover of about 5 cm, and 32ϕ26 longitudinal B 450 C bars (5ϕ26 bars along each side and 3ϕ26 in each corner of the square section), it can be easily shown that the column remains substantially linear. In fact, whilst in the case of pure flexure the yielding moment may be roughly estimated as $M_y = \left(13 \cdot 531\text{mm}^2 \cdot 450\frac{\text{N}}{\text{mm}^2}\right) \cdot (0.9 \cdot 0.75\text{m}) \cong 2100\text{kNm}$, in the case of *N*-*M* combined action a compressive force is beneficial and increases the flexural resistance (e.g. 2800 kNm for $N = 3000$ kN, and 2600 kNm for the most unfavourable case of $N = 1000$ kN).

12. Conclusions

In this paper, an application of the procedure for the seismic design of building structures equipped with viscous dampers proposed by Silvestri *et al*. (2010) has been carried out with reference to the case-study of a 2-storey precast reinforced concrete shopping mall built in northern Italy. It is found that the proposed procedure allows an easy identification of the mechanical characteristics of the viscous dampers capable of providing the damped structure with the target levels of performances (i.e. reduction in seismic response with respect to the reference structure with no additional damping system).

Further, the case-study here presented also compares the use of viscous dampers (under various configurations) with the use of traditional (steel) bracing systems and buckling restrained braces. The results of the numerical time-history simulations show that traditional bracing systems (made up using steel elements) are capable of satisfying the deformability requirements (the main issue in the seismic design of precast structures) at the expense of large forces both upon the foundations and in the bracing system. On the other hand, the insertion of viscous dampers (following either a

Fixed-Point placement or an Inter-Storey placement) allows, at the same time, to reduce the drifts of the structure within the limitations provided by the codes and to keep the forces in the bracing system and upon the foundations at acceptable levels.

Dedication

This manuscript is dedicated to the memory of our Prof. Claudio Ceccoli, who, both as person and as teacher, has always inspired and guided our research works with his deep theoretical knowledge and large professional experience, and still does.

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