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IMPROVEMENT OF THE SEISMIC PERFORMANCE OF R.C. FRAMED BUILDINGS WITH SOFT STOREYS BY HYSTERETIC DAMPED BRACES

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Abstract

A suitable insertion of steel braces equipped with dampers can be very effective in retrofitting of r.c. framed buildings in order to attain a designated seismic performance for a given intensity of the ground motion. In particular, this represents an easy and less expensive solution, in comparison with other retrofitting techniques, to overcome the vulnerability problems resulting from an irregular layout of masonry infill walls as in the case of soft storeys. Currently a wide variety of energy dissipating devices (e.g. friction, metallic, viscous or viscoelastic dampers) are available. In this paper hysteretic (metallic) dampers are considered, but, with a suitable modification, the results can be extended to other kinds of damping devices.

Current seismic codes allow for the use of the above devices but few codes provide simplified design. A Displacement-Based Design (DBD) procedure, in which the design starts from a target deformation of an equivalent elastic linear system with effective properties (i.e. secant stiffness and equivalent viscous damping), is proposed in the present work for the seismic retrofitting of a r.c. framed structure with soft storeys. Hysteretic damping due to nonlinear behaviour of the masonry infills (MIs) is taken into account. The criteria for proportioning the hysteretic damped braces (HYDBs) are aimed to obtain a damped braced structure which can be considered globally regular with regard to stiffness. In detail, the stiffness distribution of HYDBs is selected assuming a same value of the drift ratio at every storey of the irregularly infilled building retrofitted by insertion of damped braces; moreover, the strength distribution of the HYDBs is assumed so that their activation tends to occur at every storey simultaneously, before reaching the yielding of the framed structure and/or a preset damage level of the infill walls.

To check the effectiveness and reliability of the criteria proposed for proportioning the HYDBs, a numerical investigation is carried out with reference to a six-storey r.c. framed building, originally designed in compliance with a former Italian code [DM 1996] for a medium-risk zone. It is supposed that, after a change in use of the building from residential to office removing the masonry infills of the first storey, the resulting soft-storey building has to be retrofitted by inserting of HYDBs to attain a performance level imposed by the current Italian code [NTC 2018] in a high-risk zone. Nonlinear dynamic analyses of the bare framed building, infilled framed building with first soft-storey and infilled framed buildings with first soft-storey retrofitted by HYDBs having different properties are carried out for a set of artificially generated ground motions whose response spectra match that adopted by NTC 2018 for the life-safety performance level. To this end, r.c. frame members are idealized by a two-component model, assuming a bilinear moment-curvature law and considering the effect of the axial load in the columns. The response of a HYDB is idealized by a bilinear law, preventing buckling. A simplified diagonal pin-jointed strut model reacting only in compression takes into account the in-plane failure modes that can occur in an infill panel (i.e., compression at the center, compression at the corners, shear sliding and diagonal tension). More precisely, the response of a diagonal strut is simulated by a trilinear law with stiffness degradation and suitable hysteretic properties. The effectiveness of the criteria for proportioning the HYDBs, on which the proposed DBD design procedure is based, is checked for different properties of the HYDBs.

Keywords: seismic retrofitting, r.c. framed structures, soft storeys, damped braces, displacement based design.

1. Introduction

Masonry infill walls (MIs) can modify significantly the stiffness, strength and mass distributions of a reinforced concrete (r.c.) framed building. Irregularities in the MIs distribution, in elevation (e.g. soft-storeys) or in plan (e.g. unsymmetrical layout), can lead to severe seismic damage (Hak *et al*. [1], Braga *et al.* [2]). To mitigate these effects, damped steel braces with suitable layout and properties can be inserted in the framed structure. A wide variety of energy dissipating devices is now available (e.g., see Soong and Dargush [3]). Current seismic codes allow for the use of these devices (e.g. Eurocode 8 [4]; Italian code (NTC 2018 [5]) but few codes provide simplified design criteria (e.g. American code FEMA 356 [6]).

In a previous work (Mazza and Vulcano [7]), a Displacement-Based Design (DBD) procedure was proposed for proportioning hysteretic damped braces (HYDBs) in order to attain, for a specific level of seismic intensity, a designated performance level of an existing regular r.c. framed structure. The DBD procedure is based on the definition of an equivalent elastic linear system with effective properties by referring to a target deformation (see Priestley *et al.* [8]). Moreover, the same ratio between the elastic stiffness of the damped braces (DBs) and that of the framed structure was assumed at every storey ("proportional stiffness criterion").

Successively, the authors (Mazza *et al.* [9, 10]) extended the DBD procedure in the case of framed buildings with irregular in-elevation distribution of the MIs, idealizing the response of a single MI by a diagonal strut model simply assuming an elastic brittle behaviour. In this way the hysteretic behaviour of the MIs was neglected. To overcome this limitation, in this paper the hysteretic behaviour of the MIs is taken into account in the design criteria for proportioning the HYDBs. Moreover, with reference to a six-storey building with first soft storey, criteria for selecting the stiffness and strength in-elevation distributions of the HYDBs are proposed aiming to recover the regularity of the building: precisely, assuming the same drift ratio at every storey (at least for elastic behaviour) and a rather uniform yielding of the hysteretic dampers at all the storeys. On the basis of the above criteria, an extension of the DBD procedure is proposed.

To check the effectiveness of the above design criteria for proportioning the HYDBs, a numerical investigation is carried out supposing that the r.c. framed building, originally designed according to a former Italian code (DM 1996 [11]) for a medium-risk zone, after a change in use of the building from residential to office removing the masonry infills of the first storey, has to be retrofitted by inserting of HYDBs to comply with NTC 2018 in a high-risk zone. Nonlinear dynamic responses of the bare framed building, infilled framed building with first soft-storey and infilled framed buildings with first soft-storey retrofitted by HYDBs having different properties, all subjected to a set of three artificial ground motions compatible with the NTC18 design spectrum for the life-safety performance level, are compared.

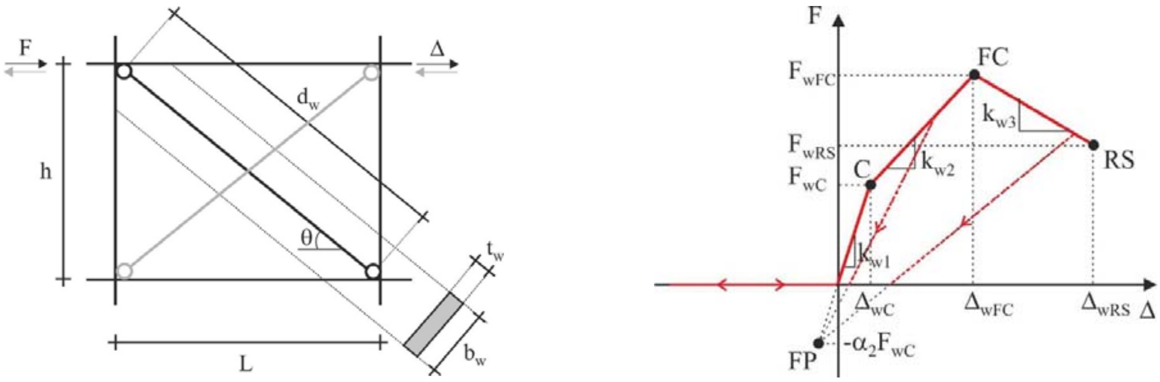
2. Nonlinear modelling of masonry infills

Before extending the DBD procedure previously proposed in Refs. [7, 9, 10] for proportioning the HYDBs, the hysteretic behaviour of the MIs is taken into account as indicated below.

Many models with different degrees of discretization and accuracy have been proposed in the literature to reproduce the in-plane nonlinear seismic behaviour of the MIs (Liberatore *et al.* [12]). Any model has its advantages and limitations, but the equivalent diagonal pin-jointed strut model without tension resistance (Figure 1a) allows obtaining the key features of the inelastic response without detailed information about local phenomena. To evaluate the width (b_w) of the diagonal strut, with length d_w and total thickness t_w, the expression proposed by Bertoldi *et al.* [13] is used:

$$b_w/d_w = K_1/(\lambda h) + K_2 \quad , \quad \lambda h = \sqrt[4]{\frac{E_{w\theta}\, t_w\, sin2\theta}{4\, E_c\, I_c\, h_w}}\, h \tag{1a,b}$$

where the dimensionless parameters K_1 and K_2 depend on the dimensionless parameter λh, originally proposed by Stafford Smith [14], h being the centreline height of a frame storey, h_w the height of the infill panel, θ the slope of the diagonal strut with respect to the horizontal direction, E_c the elastic modulus of concrete, I_c the momentum of inertia of the columns and $E_{w\theta}$ the equivalent modulus of the infill panel depending on the secant moduli of elasticity in the horizontal and vertical directions (E_{wh} and E_{wv}, respectively), G is the shear modulus and ν the Poisson ratio. Further details can be found in [15].



(a) Strut model (b) Pivot hysteretic model

Figure 1 - Nonlinear modelling of a masonry infill panel

The diagonal strut model can take into account the in-plane failure modes that can occur in the infill panels when subjected to seismic loading. Four failure modes are considered, with the corresponding equivalent compressive strengths for diagonal compression (σ_{w1}), crushing in the corners in contact with the frame (σ_{w2}), sliding shear along horizontal joints (σ_{w3}) and diagonal tension (σ_{w4}) [13]:

$$\sigma_{w1} = \frac{1.16 f_{wv}\, tan\theta}{K_1 + K_2\, \lambda h} \quad ; \quad \sigma_{w2} = \frac{1.12\, f_{wv}\, sin\theta\, cos\theta}{K_1 (\lambda h)^{-0.12} + K_2 (\lambda h)^{0.88}} \qquad (2a, b)$$

$$\sigma_{w3} = \frac{(1.2 sin\theta + 0.45 cos\theta) f_{wu} + 0.3\sigma_v}{b_w/d_w} \quad ; \quad \sigma_{w4} = \frac{0.6 sin\theta f_{ws} + 0.3\sigma_v}{b_w/d_w} \qquad (2c, d)$$

where f_{wv} is the compression strength in the vertical direction, f_{wu} is the slide resistance of the mortar joints, f_{ws} is the shear resistance under diagonal compression, σ_v is the vertical compression stress due to gravity loads. Then, the maximum lateral strength of the strut is evaluated as

$$F_w = \sigma_{w,min}\, b_w\, t_w\, cos\theta \quad , \quad \sigma_{w,min} = \min\{\sigma_{w1}, \sigma_{w2}, \sigma_{w3}, \sigma_{w4}\} \qquad (3a, b)$$

As shown in Figure 1b, the skeleton curve of the lateral force-interstorey drift law (F-Δ) is described by three linear branches, which can be defined by parameters α, β and ξ (Cavaleri *et al.* [16]). In detail, the first ascending branch corresponds to the uncracked stage, until the attainment of point C identified by

$$F_{wC} = \alpha F_w \;,\quad \alpha = 0.4 \quad ; \quad \Delta_{wC} = F_{wC}/k_{w1} \quad ; \quad k_{w1} = E_{w\theta}\, b_w\, t_w\, cos^2\theta / d_w \qquad (4a, b, c)$$

The second ascending branch represents the post-cracking phase up to the attainment of point FC, corresponding to the full development of the cracking:

$$F_{wFC} = F_w \,;\; \Delta_{wFC} = \Delta_{wC} + \frac{(F_{wFC} - F_{wC})}{k_{w2}} \quad ; \quad k_{w2} = \beta k_{w1} \;,\; \beta = 0.15 \qquad (5a, b, c)$$

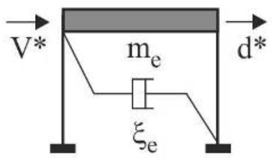
The third descending branch describes the post-peak strength deterioration of the infill up to the attainment of point RS:

$$F_{wRS} = 0.7 F_{wFC} \,; \quad \Delta_{wRS} = \frac{1}{\xi} ln\left(\frac{F_{wFC}}{F_{wRS}} e^{\xi \Delta_{wFC}}\right) \;,\; \xi = 0.02 \;; \quad k_{w3} = tan\left(\frac{F_{wFC} - F_{wRS}}{\Delta_{wFC} - \Delta_{wRS}}\right) \qquad (6a, b, c)$$

The pivot hysteretic model in Figure 1b simulates the nonlinear force-displacement law of the equivalent diagonal strut, based on geometrical rules that define loading and unloading branches. In detail, once the strength envelope, without tension resistance, is defined, the cyclic behaviour can be simply described as suggested by Cavaleri and Di Trapani [17], referring to a fundamental pivot point (FP) depending on the cracking resistance (F_{wC}) reduced by the parameter α_2(=0.25).

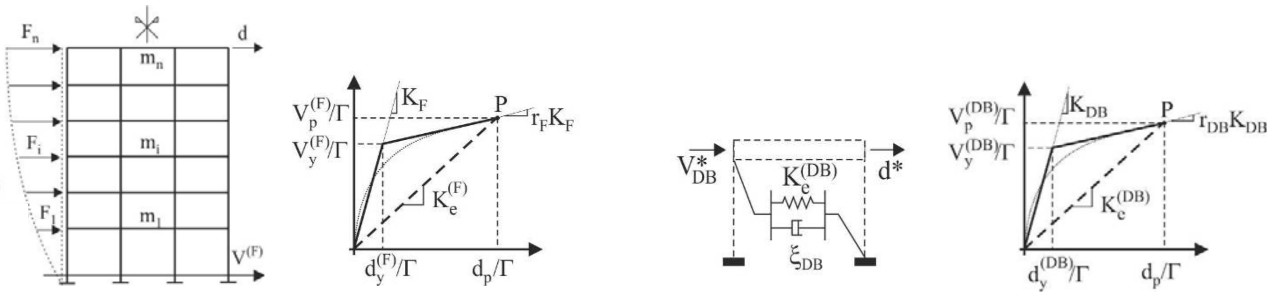
3. Proposed design procedure for proportioning the hysteretic damped braces

The main assumptions on which the design procedure proposed in this paper for proportioning the HYDBs is based are briefly illustrated below. Further detail can be found in references [7, 9, 10, 15] mentioned above. More precisely, the design procedure is based on the definition of an equivalent single degree of freedom (ESDOF) system (Figure 2a), which can idealize the nonlinear behaviour of the entire structural system (e.g., an infilled framed structure with damped braces) or that of the single systems acting in parallel (i.e., the bare framed structure, the infill panels and the damped braces). In particular, the pushover analysis of a given frame (Figure 2b) allows to idealize as bilinear the base shear-top displacement curve ($V^{(F)}$-d) and, then, to define the corresponding ESDOF system for which the equivalent viscous damping due to hysteresis ($\xi_F^{(h)}$) can be evaluated as depending on the effective ductility level ($\mu_F=d_p/d_y^{(F)}$; d_p=performance displacement) and the strain hardening ratio r_F (see [7]). Analogously, the equivalent viscous damping due to hysteresis of the HYDBs (ξ_{DB}) can be evaluated as depending on the effective ductility level ($\mu_{DB}=d_p/d_y^{(DB)}$) and the strain hardening ratio r_{DB} once an idealized bilinear shear-displacement curve ($V^{(DB)}$-$d^{(DB)}$) is assumed (Figure 2c).



$$d^*=d\,/\,\Gamma$$
$$V^*=V\,/\,\Gamma$$
$$\Gamma=\sum m_i\phi_i\Big/\sum m_i\phi_i^2$$
$$m_e=\sum m_i\phi_i\ (\phi_n=1)$$
$$T_e=2\pi\sqrt{m_e/K_e}$$

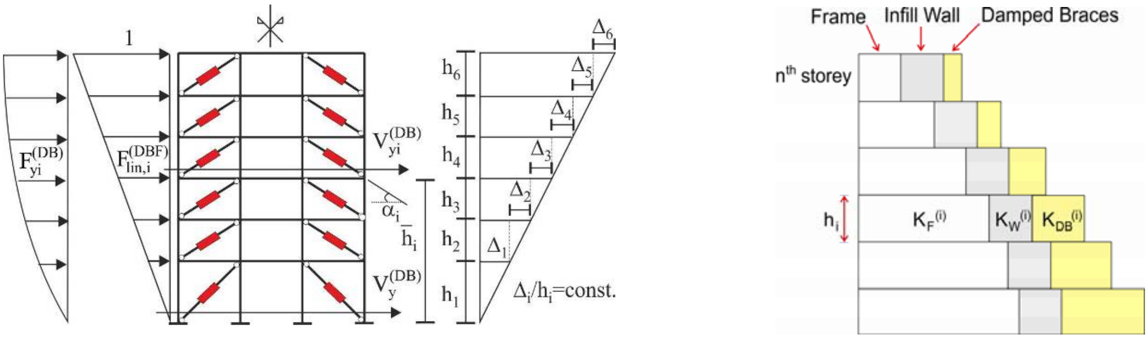
(a) Properties: d=displacement; V=shear; $\phi_{i=1,n}$=displacement shape; m_e=effective mass; T_e=effective period



(b) Given frame (F)　　(c) Damped braces (DBs)

Figure 2 - Equivalent single degree of freedom (ESDOF) systems

The design of the HYDBs is based on suitable assumptions on their stiffness and strength distributions. As done in references [9, 10, 15], aiming to obtain an infilled framed structure with damped braces (IFDB) globally regular in stiffness, a constant value of the drift ratio, i.e. the ratio of the interstorey drift to the storey height (=Δ_i/h_i), is assumed. For simplicity in Figure 3a the case of a damped braced frame (DBF) is shown. It should be noted that often in practice a framed structure is designed neglecting the MIs contribution, but in reality the MIs modify the stress distribution in the frame members. Indeed, even in case of MIs regularly distributed in elevation whose contribution could be considered comparable at every storey, the increase of stiffness and strength due to MIs, respect to the analogous properties of the bare framed structure, is generally more marked towards the upper storeys (Figure 3b). Therefore, to compensate this increase it is reasonable inserting HYDBs more stiff and strong in a lower storey rather than in an upper storey; even more in case of a soft storey, where it is necessary to make up for the lack of the infill wall contribution.



(a) HYDBs in a damped braced frame (DBF) (b) Storey stiffness distributions in an IFDB

Figure 3 – Assumptions regarding the stiffness distribution of the HYDBs

Aiming to limit not only the plastic deformations in the frame members, but mostly to get a limited and rather uniform damage of the MIs and other nonstructural parts, it can be considered suitable for the HYDB design assuming a sufficiently limited value of the drift ratio at every storey of the IFDB (i.e., Δ_i/h_i=constant as in Figure 3a). Then, with reference to two generic storeys (i, j), it can be written

$$\frac{\Delta_i}{h_i} = \frac{\Delta_j}{h_j} \ ; \ \Delta_i = \frac{V_T^{(i)}}{K_T^{(i)}} \ ; \ \Delta_j = \frac{V_T^{(j)}}{K_T^{(j)}} \qquad \text{(7a, b, c)}$$

where $V_T^{(i,j)}$ and $K_T^{(i,j)}$ are, respectively, the total (elastic) shear and total lateral stiffness at the two considered storeys of an IFDB. In particular, for a generic (i^{th}) storey, it can be written

$$V_T^{(i)} = V_F^{(i)} + V_W^{(i)} + V_{DB}^{(i)} \ ; \qquad K_T^{(i)} = K_F^{(i)} + K_W^{(i)} + K_{DB}^{(i)} \qquad \text{(8a, b)}$$

$V_F^{(i)}$, $V_W^{(i)}$, $V_{DB}^{(i)}$ being the storey shear contributions of the bare frame, infilled walls and damped braces, respectively, while $K_F^{(i)}$, $K_W^{(i)}$, $K_{DB}^{(i)}$ are the corresponding contributions to the storey lateral stiffness.

According to Equations (7), the total stiffness at the i^{th} storey can be expressed by the properties of the top (n^{th}) storey ($K_T^{(n)}$, $V_T^{(n)}$, h_n):

$$K_T^{(i)} = K_T^{(n)} \frac{V_T^{(i)}}{V_T^{(n)}} \frac{h_n}{h_i} \qquad (9)$$

where the total stiffness of the top storey, $K_T^{(n)}$, can be obtained, according to Equation (8b), as follows:

$$K_T^{(n)} = K_{IF}^{(n)} + K_{DB}^{(n)} = (1+\alpha_n)\, K_{IF}^{(n)} \qquad (10)$$

where

$$K_{IF}^{(n)} = K_F^{(n)} + K_W^{(n)}; \quad \alpha_n = \frac{K_{DB}^{(n)}}{K_{IF}^{(n)}} \qquad \text{(11b, c)}$$

$K_{IF}^{(n)}$ being the lateral stiffness of the infilled framed building at the top storey and α_n a dimensionless parameter, which, although referred to the top storey, allows to identify the stiffness level at every storey.

Indeed, Equations (9) and (10) give the total lateral stiffness at a generic storey:

$$K_T^{(i)} = (1+\alpha_n)\, K_{IF}^{(n)} \frac{V_T^{(i)}}{V_T^{(n)}} \frac{h_n}{h_i} \qquad (12)$$

Then, according to Equation (8b), the lateral stiffness of the damped braces at the same storey, $K_{DB}^{(i)}$, is obtained as

$$K_{DB}^{(i)} = K_T^{(i)} - K_{IF}^{(i)}; \quad K_{IF}^{(i)} = K_F^{(i)} + K_W^{(i)} \tag{13a,b}$$

where $K_F^{(i)}$ and $K_W^{(i)}$ can be calculated, in accordance with the above assumption, for the lateral loading pattern inducing the same drift ratio at every storey of the framed structure and the infill walls, respectively. For the evaluation of each one of these stiffnesses it may be assumed a suitable value of the secant stiffness (i.e. reducing the initial stiffness of the frame members and infill panels). It should be noted that a strictly necessary stiffness of the HYDBs corresponds to assume α_n=0 (i.e., no damped braces at the top storey).

With regard to the strength distribution, it can be assumed that the HYDBs yielding occurs at all the storeys (for a same suitable drift ratio) before yielding of the framed structure and/or reaching an accepted damage level of the MIs.

On the basis of the above assumptions, the main steps for proportioning the HYDBs in case of in-elevation irregularity of the MIs (giving rise to soft storeys) can be summarized as indicated below, taking into account the nonlinear behaviour of the MIs as specified in Section 2.

Step 1. The pushover analysis is carried out for the infilled framed structure (IF). Because the hysteretic behaviour of the framed structure (F) and that of the infill walls (W) is different, it is advisable idealizing separately the curve ($V^{(F)}$-d) and the analogous one for the infill walls ($V^{(W)}$-d). This allows to evaluate the equivalent stiffnesses of the frame structure ($K_e^{(F)}$) and the infill walls ($K_e^{(W)}$) as well as the equivalent viscous damping factors due to hysteresis of the framed structure ($\xi_F^{(h)}$) and the infill walls (ξ_W). In particular, the latter factor needs a suitable calibration according to the pivot model (Figure 1b) to be considered in both the loading directions.

Step 2. Evaluation of the equivalent viscous damping due to hysteresis of the damped braces (ξ_{DB}), depending on the effective ductility level ($\mu_{DB}=d_p/d_y^{(DB)}$) and the strain hardening ratio r_{DB} once a bilinear shear-displacement curve ($V^{(DB)}$-$d^{(DB)}$) is assumed (Figure 2c).

Step 3. The equivalent viscous damping of IFDB (ξ_e) is evaluated accounting also for the equivalent viscous damping (ξ_W) and the shear contribution ($V_p^{(W)}$) of the infill walls with reference to the performance point. For this purpose the ESDOF systems representing the framed structure, the infill walls and the damped braces are considered as systems acting in parallel:

$$\xi_e = \xi_V + \frac{\xi_F^{(h)} V_p^{(F)} + \xi_W V_p^{(W)} + \xi_{DB} V_p^{(DB)}}{V_p^{(F)} + V_p^{(W)} + V_p^{(DB)}} \tag{14}$$

where ξ_V is a suitable value of the elastic viscous damping for the framed structure (e.g. ξ_V=5%), $\xi_F^{(h)}$ and ξ_{DB} have been calculated in steps 1 and 2, respectively, while $V_p^{(F)}$, $V_p^{(W)}$ and $V_p^{(DB)}$ represent the base-shear contributions of the bare framed structure, the infill walls and the hysteretic damped braces, respectively, all referred to the performance point P.

Step 4. The effective stiffness of the equivalent damped brace ($K_e^{(DB)}$) for retrofitting an (irregularly) infilled framed structure is calculated as

$$K_e^{(DB)} = K_e - K_e^{(F)} - K_e^{(W)}; \qquad K_e = 4\pi^2 m_e / T_e^2 \tag{15a, b}$$

where K_e, m_e and T_e are, respectively, effective stiffness, effective mass and effective period of IFDB, while $K_e^{(F)}$ and $K_e^{(W)}$ are the effective stiffnesses of the framed structure and infill walls (see step 1). In particular, T_e is evaluated by the displacement design spectrum for ξ_e once a d_p value is fixed.

Step 5. Evaluation of the stiffness properties of the HYDBs at every storey (i.e. $K_{DB}^{(i)}$) after determining, according to Equations (12) and (13), the α_n value such that the following equation, derived from the above assumptions, be satisfied (see Figures 2c and 3):

$$K_{DB} = K_{DB}^{(1)} h_1 / \sum_{i=1}^{n} h_i \tag{16}$$

Step 6. Finally, the yield shear contribution of the damped braces at the i^{th} storey can be obtained as

$$V_{DB,y}^{(i)} = K_{DB}^{(i)} \Delta_y^{(i)} \tag{17}$$

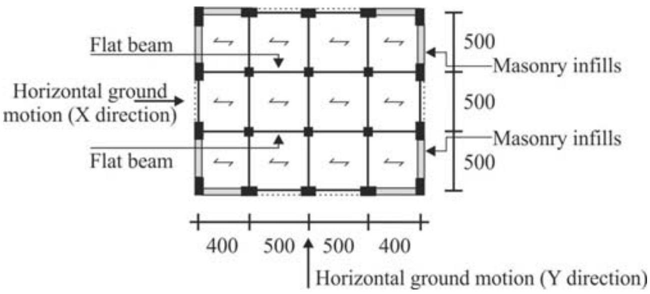
where $\Delta_y^{(i)}$ can be assumed as to be less than (or even close to) the storey drift corresponding to the first yielding of the framed part of the building and/or to an acceptable damage level of the infilled walls (or other nonstructural parts).

4. Test structures

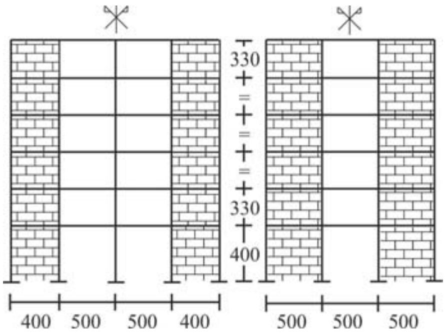
4.1 Original building

Before implementing the design procedure proposed above, it is considered very important to check the effectiveness of the criteria illustrated above for selecting the stiffness and strength distributions of the HYDBs. For this purpose a six-storey residential building with r.c. framed structure is considered as primary structure (Figure 4). This structure is designed considering the MIs as nonstructural elements regularly distributed in plan and in elevation. In Figure 4 only MIs in the corner bays of the perimeter frames are shown, supposing that the stiffness and strength contribution of the other MIs with openings (supposed rather large) is neglected. In any case the mass of all the MIs is taken into account.

The design of the original framed building is simulated in accordance with a former Italian code [11], assuming a medium-risk seismic region (seismic coefficient: C=0.07) and a typical subsoil class (main coefficients: R=ε=β=1). Any masonry infill consists of two leaves of clay horizontal hollowed bricks, with a thickness of t_{we}=12 cm (exterior leave) and t_{wi}=8 cm (interior leave), so that the total thickness is t_w=t_{we}+t_{wi}=20 cm. Skeleton curves of the MIs, for the six storeys of the original building are plotted in Figure 5 with reference to the Y direction considered in this study. Further details regarding the properties and design of the infilled framed structure can be found in references [9, 10, 15].



(a) Plan



(b) Infilled perimeter frames (IF)

Figure 4 - Original infilled framed building (units in cm)

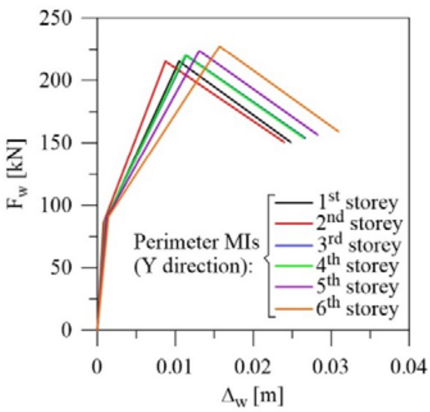


Figure 5 - Skeleton curves for the masonry infills (MIs) of the original building in the Y direction

4.2 Soft-storey building retrofitted by hysteretic damped braces

To simulate an irregularity in elevation, it is supposed that, due to a change in use of the building in Figure 4, from residential to office, the masonry infill walls of the first storey are removed, leading to a first soft storey framed building (IF_SS(1), Figure 6).

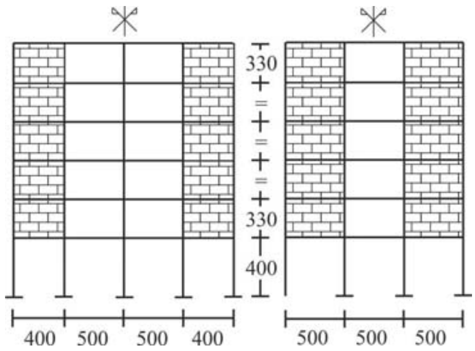
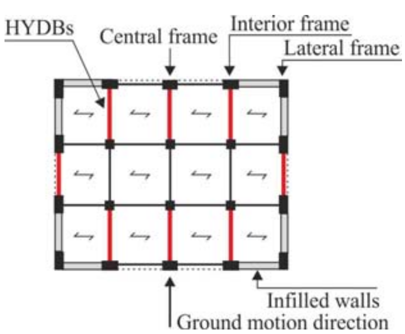


Figure 6 - Soft first storey frames (IF_SS(1) building) after the change in use (dimensions in cm)

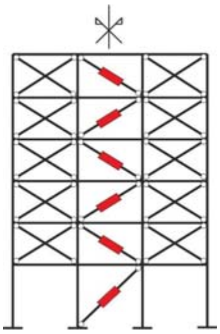
The stiffness and strength contributions of the masonry infill walls in the upper five storeys are considered idealizing any infill panel by two diagonal equivalent struts (Mainstone [18]). To upgrade the IF_SS(1) building from a medium- to a high-risk seismic region, diagonal steel braces with hysteretic dampers (HYDBs) are inserted at every storey. For simplicity, in Figure 7 only idealized MIs and HYDBs in the frames along the considered ground motion direction Y are shown.

The HYDBs are designed taking into account the contribution of the MIs by the criteria proposed in Section 3. As an example, two values of the parameter α_n (n=6) are assumed (see Equation (11c)), i.e. α_6=0 and α_6=1. With reference to these two values, the total lateral (elastic) stiffnesses of the bare framed structure ($K_F^{(i)}$), infill walls ($K_W^{(i)}$) and hysteretic damped braces ($K_{DB}^{(i)}$), together with the total yield shear contribution of the hysteretic damped braces ($V_{DB,y}^{(i)}$) at every storey, are reported in Table 1.

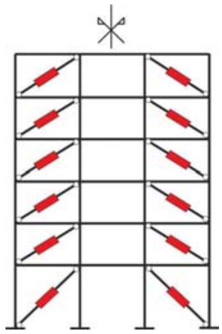
It should be noted that $K_F^{(i)}$ has been evaluated with reference to the lateral load distribution which induces the same drift ratio at every storey considering also axial and shear deformability of the frame members. For simplicity, an average value of the secant stiffness $K_W^{(i)}$ at a suitable value of the drift ratio has been assumed for the infill walls of all the upper five storeys (where MIs are still present after the change in use) and the same properties are assumed for each one of the eight diagonal damped braces at a storey (i.e. $K_{DB}^{(i)}/8$ and $V_{DB,y}^{(i)}/8$; see Figure 7). The $V_{DB,y}^{(i)}$ value has been selected assuming for all the HYDBs a yield drift ratio of 0.15%, which is less than (but close to) the value corresponding to the first yielding of the framed structure and can be considered low enough to ensure an acceptable damage of the infill walls.



(a) Plan



(b) Lateral frame with masonry infills and HYDBs



(c) Interior and central frames with HYDBs

Figure 7 - Irregularly-infilled structure retrofitted by HYDBs (IFDB) and its modelling

Table 1 - Properties of the framed structure (F), infill walls (W) and hysteretic damped braces (DB)

(a) Case of α_6=0 (IFDB_K0 building)

Storey	$K_F^{(i)}$ (kN/mm)	$K_W^{(i)}$ (kN/mm)	$K_{DB}^{(i)}$ (kN/mm)	$V_{DB,y}^{(i)}$ (kN)
6	74.735	83.2	0.	0.
5	120.583	83.2	151.142	748.154
4	195.291	83.2	244.172	1208.651
3	225.657	83.2	345.566	1710.554
2	405.025	83.2	266.329	1318.328
1	390.182	-----	280.472	1682.830

(b) Case of α_6=1 (IFDB_K1 building)

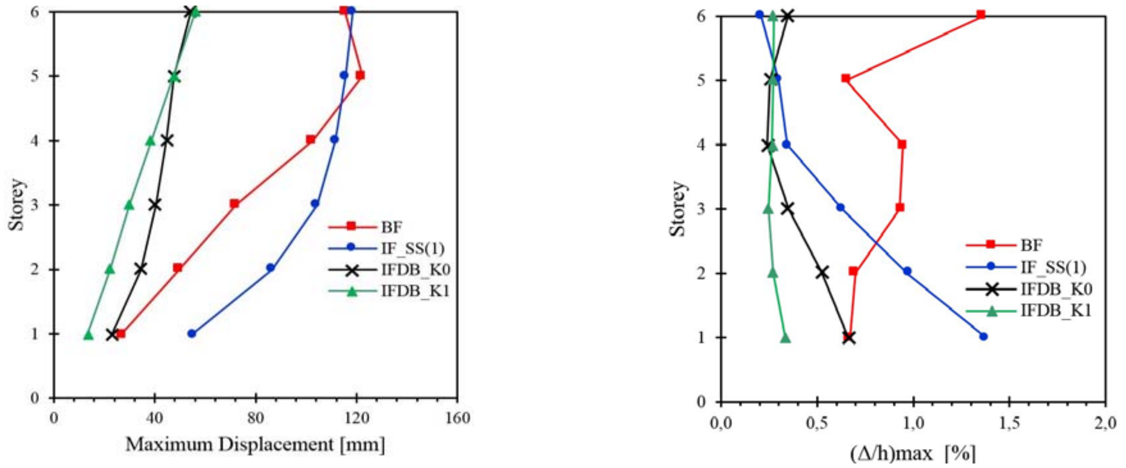
Storey	$K_F^{(i)}$ (kN/mm)	$K_W^{(i)}$ (kN/mm)	$K_{DB}^{(i)}$ (kN/mm)	$V_{DB,y}^{(i)}$ (kN)
6	74.735	106.6	181.335	897.608
5	120.583	106.6	587.841	2909.811
4	195.291	106.6	898.313	4446.648
3	225.657	106.6	1170.512	5794.033
2	405.025	106.6	1221.075	6044.323
1	390.182	-----	1149.856	6899.138

5. Numerical results

As said above, it is considered of primary importance to check the effectiveness of the design criteria illustrated in Section 3 for proportioning the HYDBs in case of in-elevation irregular layout of the MIs. For this purpose, a numerical investigation is carried out considering also the hysteretic behaviour of the MIs. The nonlinear dynamic responses of the bare framed building (BF), infilled framed building with first soft-storey (IF_SS(1); see Figure 6) and infilled framed buildings with first soft-storey retrofitted by HYDBs having different properties (IFDB_K0 and IFDB_K1; see Table 1), when subjected to a set of three artificial ground motions, are compared. Specifically, three artificial motions (duration 20 s) were generated by the SIMQKE code (Gasparini and Vanmarcke [19]) matching the NTC18 design spetrum for the life-safety state, SLV, assuming data leading to a peak ground acceleration PGA=0.423g: i.e., a_g=0.307g (reference peak acceleration), subsoil class C, building use class III, nominal life of the building V_N=70 ys.

The nonlinear dynamic analyses are carried by SAP2000 (CSI Computers and Structures [20]), assuming the floor slabs to be infinitely rigid on their own plane. Moreover, the r.c. frame members are idealized by a lumped plasticity model accounting for the effect of the axial load on the ultimate moment. As mentioned above, every infill wall is represented as a pair of equivalent diagonal struts connecting the frame joints (Figure 7b), whose response is simulated by the pivot hysteretic model (Figure 1b). The response of the HYDBs is simulated by a bilinear model assuming a strain hardening percentage of 1%. A value of 5% is assumed for the elastic viscous damping ratio of the framed structure, ξ_V. All the following results are obtained as an average of maximum values attained for the set of artificial motions.

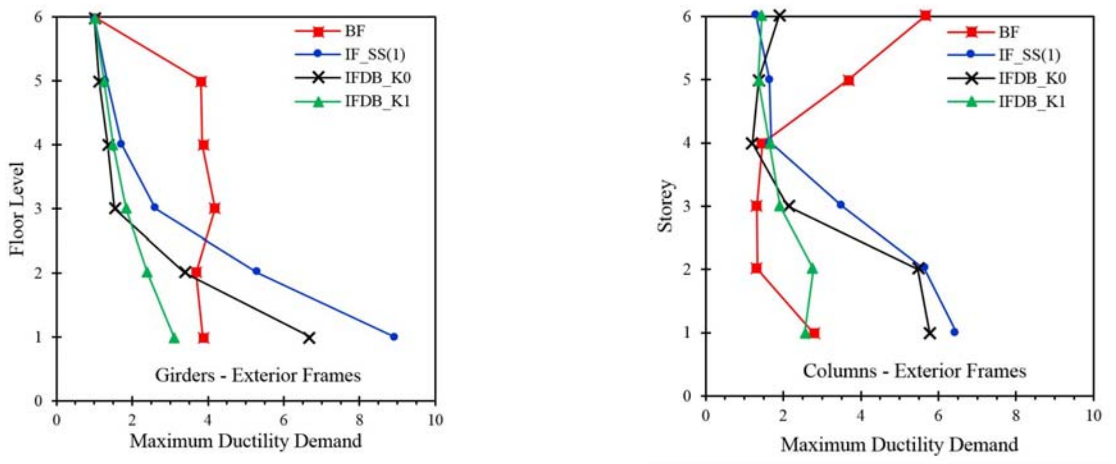
The maximum floor displacement and the maximum inter-storey drift ratio of the four buildings considered above are compared in Figure 8a and Figure 8b, respectively. As shown in Figure 8a, the IF_SS(1) building generally exhibts the largest floor displacements, especially at the lower storeys due to the large drift at the first storey of about 1.37% (Figure 8b), but its floor displacements become comparable to those of the BF building at the upper three storeys. However, the BF building shows the largest drift at the upper four storeys. The beneficial effect of the HYDBs is evident, especially for the IFDB_K1 building, exhibiting an almost linear curve for the maximum floor displacements and a rather uniform and low drift ratio (less than 0.34%).



(a) Maximum floor displacements (b) Maximum inter-storey drift ratios

Figure 8 - Response of the test structures

Ductility demands to girders and columns of the exterior frames are shown in Figure 9a and Figure 9b, respectively. Analogous results, omitted for brevity, have been obtained for the girders and columns of the interior frames [15]. As shown, for both the girders and columns the ductility demands to BF structure are larger at the upper storeys (except top-floor girders) and greater than those for the other structures, which unlike BF structure exhibit larger values at the lower storeys. The beneficial effect of the HYDBs, more marked in the lower storeys of IFDB_K1 structure rather than in IFDB_K0 structure, which undergo some detrimental effect of the first soft storey. In all the cases, the ductility demand is rather uniform and limited for IFDB_K1 structure.



(a) Girders (b) Columns

Figure 9 - Maximum ductility demand to r.c. members of the exterior frames

Finally, the curves representing the ductility demand to the HYDBs at all the storeys of the retrofitted buildings are compared in Figure 10. It is evident that the ductility demand in IFDB_K1 structure is rather uniform and limited at all the storeys, exhibiting values lower than those in IFDB_K0 structure (i.e., that without HYDBs at the top storey). A more limited plastic excursion of the HYDBs of IFDB_K1 structure, in comparison with the HYDBs of IFDB_K0 structure, was pointed out also in a previous work by the authors [15].

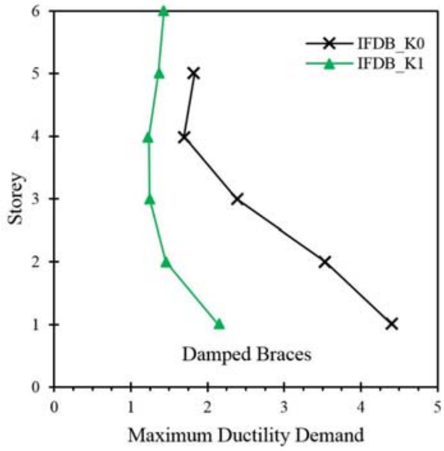


Figure 10 - Ductility demand to HYDBs of the retrofitted buildings

6. Conclusions

A DBD design procedure has been proposed to proportion HYDBs for the sesmic retrofitting of r.c. framed buildings with in-elevation irregular layout of the masonry infills. This procedure is based on design criteria aiming to obtain the same drift ratio at every storey. To check the effectiveness of the above criteria in the case study of a r.c. six-storey building, the nonlinear seismic responses of the bare framed building (BF), first soft-storey building (IF_SS(1)) and first soft-storey building retrofitted by HYDBs having different properties (IFDB_K0 and IFDB_K1), all subjected to a set of artificially generated motions, have been compared. In detail, two different stiffness levels of the HYDBs have been assumed: i.e. α_6=0 (IFDB_K0 structure, without HYDBs at the top storey) and α_6=1 (IFDB_K1 structure, having HYDBs with the same lateral stiffness of the infilled frame at the top storey).

The results have shown that the selection of HYDBs having strictly necessary stiffness (i.e. α_6=0) allows to control the response of the soft-storey building reducing floor displacements, drifts and ductility demand to the r.c. frame members. On the other hand, to limit considerably these response parameters and get a rather uniform in-elevation distribution of the drift ratio and ductility demand to r.c. members and HYDBs, it is necessary adopting rather rigid and strong HYDBs (e.g., assuming α_6=1). But this can lead to a high variation of the axial forces in the columns. In such a case it can be advisable to distribute in many frames the total stiffness and strength required to the HYDBs at every storey.

Even though the present study clarified important aspects in proportioning the HYDBs, further information can be obtained extending the parametric study to different cases of irregularly infilled buildings subjected to real ground motions with intensity corresponding to different performance levels. Moreover, to improve the reliability of the DBD procedure, a suitable calibration of the secant stiffness of the frame members and infill walls, as well as of the equivalent damping factors ($\xi_F^{(h)}$, ξ_W, ξ_{DB}), is needed.

7. Acknowledgements

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