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A DESIGN PROCEDURE OF DISSIPATIVE BRACES FOR SEISMIC UPGRADING STRUCTURES

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ABSTRACT

The research presented in this paper deals with the seismic retrofitting of existing frame structures by means of passive energy dissipation. An iterative displacement-based procedure, based on capacity spectrum is described and some applications are discussed. The procedure can be used with any typology of dissipative device and for different performance targets. In this work the procedure has been applied to an existing r.c. frame building, which shows both vertical and plan irregularities. In the application presented the buckling restrained braces have been used in order to prevent damages to both the structure and non structural elements. The evaluation of commonly Nonlinear Static Procedures (NSPs) for seismic response of the existing and retrofitted structure is presented to check the suitability of the use of NSP in the design procedure: the use of conversional NSPs to be not suitable for the case of irregular building but, once this building is retrofitted, and therefore regularized, with a bracing system, the use of NSPs for seismic response of the braced structure is effective.

INTRODUCTION

The use of dissipative bracings, though it seems conceptually clear and simple, requires a more complex design procedure than other retrofitting methods like base isolation. This greater complexity derives from the non linear behaviour of the dissipative devices and therefore of the final retrofitted structure. Despite that, during the last years, many design procedure has been published and, between those, the most useful for practical use seem to be those that are based on the Capacity Spectrum Method (CSM). In fact with this approach non linear dynamic analyses can be skipped in favor of static non linear analyses that are simpler to be managed. Otherwise, also within those procedures, many have a theoretical approach that can be difficulty associated with a
widespread professional use. In fact, frequently, the characteristics of an existing building (e.g. non regular distribution of masses and stiffness, presence of a soft story) can compromise the effectiveness of procedures that impose a predefined loading pattern during pushover analyses.

A displacement-based procedure to design dissipative bracings for the seismic protection of frame structures is proposed by *Bergami & Nuti* (2013). The procedure is based on the displacement based design using the capacity spectrum method; no dynamic non linear analyses are needed. Two performance objectives have been considered developing the procedure: protect the structure against structural damage or collapse and avoid non-structural damage as well as excessive base shear. The compliance is obtained dimensioning dissipative braces to limit global displacements and interstorey drifts. The design of dissipative devices has two main goals: improve dissipation and regularize

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strength end stiffness distribution (this can be done adopting an adequate criterion to distribute the braces along the elevation and inside the plan of the building). To check such hypothesis of the design procedure, the first aim of this paper is to apply this design procedure of dissipative braces to a medium rise existing r.c. building, which presents both vertical and plan irregularities, and improvement of braces in torsion effects.

During the design procedure, Nonlinear Static Procedure (NSP) is adopted to evaluate the seismic response of the existing and retrofitted structure. Few studies focused on the extension of NSPs to the case of braced irregular structures, therefore the second aim of this paper to check the whether the commonly used NSPs can be used to evaluate the seismic response for the braced structure.

To these purpose, after brief description, the design procedure of dissipative braces is applied to an existing irregular structure to protect the structure against structural damage during given seismic event. Comparison of the results obtained with nonlinear dynamic analysis, which is taken as "exact" results, enables the evaluation of the accuracy of the different NSPs on the braced structure to check the effectiveness of the design procedure

DISSIPATIVE BRACING POSITIONS: STRUCTURAL EFFECTS

The insertion of dissipative braces into the structural frame involves significant effects that can be grouped in two categories: effects on structural response and effects on the architecture of the building. Concerning the former the braces increase both stiffness and strength and consequently, as usually happens, both modal shapes and the capacity curve of the structure are modified. Moreover, for a given top displacement, these improve damping and, therefore, reduce demand. In this respect stiffness increase could render less efficient, or even useless, the increase of dissipation. Therefore a careful mix of stiffness and dissipation is requested: this subject is discussed in the following.

The bracing system has to be compatible with the architecture of the building: therefore spatial distribution of the braces descents from a compromise between the optimization of the dissipative system and the functionality of the building.

Although braces distribution should be analyzed case by case some general considerations can be made: braces should reduce or eliminate eventual translation-rotation coupling effects, in-duce constant interstorey drifts, exclude soft storey behavior and maximize damping for a given top displacement.

Different criteria to distribute the additional stiffness are proposed in scientific literature: constant at each story, proportional to story shear, proportional to interstorey drifts of the original structure. In this work the latter is assumed and therefore, given the interstorey drift δ_j, the stiffness $K'_{b,j}$ corresponding to each storey of the bracing system is:

$$K'_{b,j} = K_{global} c_{b,j} \tag{1}$$

where

$$c_{bj} = \frac{\delta_j}{\max_j \{\delta_j\}} \tag{2}$$

Each brace is a composite element realized coupling an elastic element (usually a steel pro-file) with a dissipative device in series. The latter will determines the desired yielding force whereas the former will be designed to assure the desired stiffness of the series.

EVALUATION OF THE EQUIVALENT VISCOUS DAMPING

A specific energy dissipated by the structure and the braces corresponds to each deformation reached by the structure, be it with or without dissipative braces; the dissipated energy can be expressed in terms of equivalent viscous damping. Referring to the formula proposed by *A.K. Chopra* (2001), the equivalent viscous damping of the structure $\nu_{eq,S}$ at the generic displacement D can be expressed as follows:

$$\nu_{eq,S} = \frac{1}{4\pi} \frac{E_{D,S}}{E_{S,S}} \tag{3}$$

All the parameters of the Eq. (3) can be easily determined from the capacity curve: is the energy dissipated in a single cycle of amplitude D and $E_{S,S}$ is the elastic strain energy corresponding to the displacement D. Referring to an equivalent bilinear capacity curve (it can be determined from the capacity curve using one of the methods available in literature) terms of Eq. (3), considering an ideal elasto-plastic hysteretic cycle, can be determined as follow:

$$E_{D,S}^{bilinear} = 4\left(F_{sy}D - D_{sy}F_s\left(D\right)\right) \tag{4}$$

$$E_{S,S} = \frac{1}{2}DF_s\left(D\right) \tag{5}$$

with:

D	the displacement reached from the structure
$F_s\left(D\right)$	the force corresponding to D (the force is the base shear)
D_{sy}	displacement at yielding
F_{sy}	the yielding force (base shear at yielding)

It is well known that the hysteretic cycle of a real structure differs from the ideal cycle, therefore this difference can be taken into account adopting a corrective coefficient cS for the structure and cB for the braces (c =1 for the ideal elasto-plastic behaviour).
Therefore:

$$E_{D,S} = \chi_S E_{D,S}^{bilinear} \tag{6}$$

$$E_{D,B} = \chi_B E_{D,B}^{bilinear} \tag{7}$$

with $E_{D,B}^{bilinear}$ the energy dissipated by the ideal hysteretic cycle of the dissipative brace.

For the applications discussed in this paper the parameter χ_S has been determined referring to the provisions of ATC40 [1996]. For the braces the assumption of $\chi_B \approx 1$ has been considered reasonable: in fact, according to AISC/SEAOC-Recommended Provisions for Buck-ling-Restrained Braced Frames [2005], the force-displacement relationship of a BRB can be idealized as a bilinear curve. However different values can be adopted, if the case, with no difference in the procedure. Authors have assumed a bilinear curve characterized by a yielding force equal to the yielding traction force (the maximum compressive strength of BRBs is slightly larger than the maximum tensile strength due to the confining effect of the external tube): the hysteretic cycle obtained is elasto-plastic but precautionary smaller than the real one. Than the evaluation of the equivalent viscous damping of the braced structure $\nu_{eq,S+\mathrm{B}}$, to be added to the inherent damping ν_I (usually $\nu_I = 5\%$ for r.c. structures and $\nu_I = 2\%$ for steel ones), can be obtained using the following expression:

$$\nu_{eq,S+B} = \frac{1}{4\pi}\frac{E_{D,S+B}}{E_{S,S+B}} = \frac{1}{4\pi}\left[\frac{\chi_S E_{D,S}^{bilinear}}{E_{S,S+B}} + \frac{\chi_B \sum_j E_{D,B,j}^{bilinear}}{E_{S,S+B}}\right] \tag{8}$$

$$\nu_{eq,S} = \chi_S \frac{1}{4\pi}\frac{E_{D,S}^{bilinear}}{E_{S,S+B}};\nu_{eq,B} = \chi_B \frac{1}{4\pi}\frac{\sum_j E_{D,B,j}^{bilinear}}{E_{S,S+B}} \tag{9}$$

where $E_{D,B,j}^{bilinear}$ is the energy dissipated by the dissipative braces placed at level j.

Eq. (8) can be generalized assuming that $E_{D,B,j}^{bilinear} = \sum_i E_{D,B,i}^{bilinear}$ with $E_{D,B,i}^{bilinear}$ the energy dissipated by the i braces placed at level j. Note that $\nu_{eq,S}$ and $\nu_{eq,\mathrm{B}}$ are obtained dividing the dissipated energy, determined from the capacity curve of S or B respectively, by the elastic strain energy of the braced structure, determined from the curve of S+B.

THE DESIGN PROCEDURE: GENERAL FORMULATION MAIN STEPS

The design procedure can be applied using every typology of pushover because it requires the only definition of capacity curve and interstorey drift distribution. Therefore the use of the multimodal procedure doesn't modify the proposed procedure that can be summarized in the following steps:

(1) Define the seismic action: the seismic action is defined in terms of elastic response acceleration spectrum (T-Sa).

(2) Select the target displacement: the target displacement is selected (for example the top displacement Dt*) according to the performance desired (limit state).

(3) Define the capacity curve: the capacity curve of the braced structure S+B, in terms of top displacement and base shear (Dt-Vb), is determined via pushover analysis. The pushover analysis can be easily performed using a software for structural analysis: many different force distributions can be adopted selecting the best option for the specific case (e.g. modal shape load profile).
If a modal shape load profile has been selected it is important to underline that the modal shape is influenced by the bracing system and consequently, at each iteration, the load profile has to be updated to the modal shape of the current braced structure.
Notice that, at the first iteration, the structure without braces is considered and therefore the capacity curve obtained will be fundamental for the evaluation of the contribution offered by the existing structure to the braced structure of the subsequent iterations.

(4) Define the equivalent bilinear capacity curve: the capacity curve is approximated by a simpler bilinear curve Dt-F_{S+B} that is completely defined by the yielding point ($D_{S+B,y}$, $F_{S+B,y}$) and the hardening ratio β_{S+B} (at the first iteration the parameters correspond to $D_{S,y}$, $F_{S,y}$, β_S of the existing building).

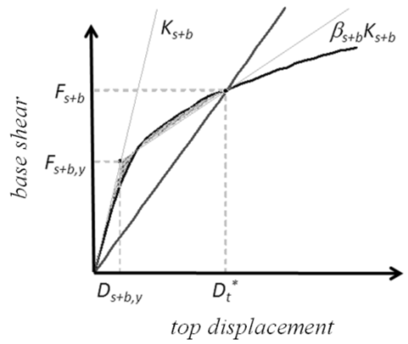

Figure 1 Evaluation of the equivalent bilinear capacity curve

(5) Define equivalent single degree of freedom: MDOF system is converted in a SDOF system by transforming the capacity curve into the capacity spectrum (S_{dt}-S_{ab})

$$S_{dt} = \frac{D_t}{\Gamma \phi_t}; S_a = \frac{F_{S+B}}{\Gamma \cdot L} \tag{10}$$

where Γ is the participation factor of the modal shape ϕ ($\Gamma = (\phi^T M I)/(\phi^T M \phi)$) and $L = (\phi^T M I)$.
The modal characteristics of the braced structure may change at every iteration due to new brace characteristics. Therefore ϕ, Γ and L have to be updated with the current configuration.

(6) Evaluate the required equivalent viscous damping: the equivalent viscous damping $\nu^*_{eq,S+B}$ of the braced structure to meet the displacement of the equivalent SDOF system and the target spectral displacement $S^*_{dt} = D^*_t / (\Gamma \phi^T)$ is determined.
According to the Capacity Spectrum Method the demand spectrum is obtained reducing the 5% damping response spectrum by multiplying for the damping correction factor η that is function of ν_{tot}

$$\eta = \sqrt{\frac{10}{5 + \nu_{tot} \cdot 100}} = \frac{S_{\nu_{eff}}}{S_{5\%}} \tag{11}$$

From Eq. (11) one obtain ν^*_{tot} the damping needed to reduce displacement up to the target S^*_{dt}.

$$v_{tot}^{*}=0.1\left(\frac{S_{5\%}}{S_{dt}^{*}}\right)^{2}-0.05 \tag{12}$$

(7) Evaluate the equivalent viscous damping contribution due to the naked structure: the contribute to damping of the structure $v^{*}_{eq,S}(D_t^{*})$ can be determined from Eq. (9) being Dt* the top displacement corresponding to $E_{D,S}^{bilinear}$ and $E_{S,S+B}$ that are the energy dissipated by S and the elastic strain energy of S+B ($E_{D,S}^{bilinear}$ and $E_{S,S+B}$ are determined from the capacity curve of S and S+B respectively).

(8) Evaluate the additional equivalent viscous damping contribution due to braces: given v_{tot}^{*} from Eq. (12) the equivalent viscous damping needed to be supplied by the braces $v^{*}_{eq,B}(D_t^{*})$ is evaluated from Eq. (8) as follows:

$$v^{*}_{eq,B}(D_t^{*})=v_{tot}^{*}(D_t^{*})-v_{eq,S}^{*}(D_t^{*})-v_I \tag{13}$$

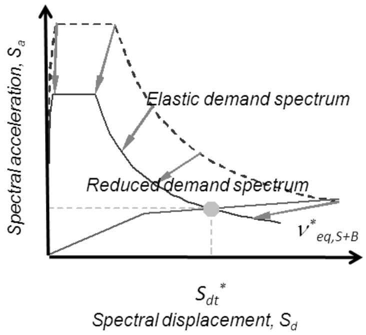

Figure 2 Evaluation of the equivalent viscous damping needed to achieve the target performance point

(9) Dimensioning of the braces: once the required equivalent viscous damping $v^{*}_{eq,B}(D_t^{*})$ has been evaluated from Eq. (13), axial stiffness and yielding strength required to achieve the desired additional damping can be determined with the same procedure previously adopted for the structure (step 7).

The energy dissipated by the braces inserted at each jth level can be expressed as:

$$E_{D,B}^{bilinear}=\sum_{j=1}^{n}4\left(F_{by}^{'}\delta_j^{'}-\delta_{y,j}^{'}F_{b,j}^{'}(\delta_j^{'})\right) \tag{14}$$

being $\delta_j^{'}$ the component of the interstory drift δ_j at j_{th} of the n floors along the axe of the brace ($\delta_{y,j}^{'}$ is the axial displacement corresponding to yielding of the device).

The axial displacement of the damping brace at the j_{th}-floor $\delta_{b,j}^{'}$ can be determined from its inclination angle $\theta_{b,j}$ and interstorey drift $\delta_j=D_j-D_{j-1}$: therefore $\delta_{b,j}^{'}=\delta_j\cos\theta_{bj}$.

The dissipative brace is usually constituted by a dissipative device (e.g. the BRB) assem-bled in series with an extension element (e.g. realized with a steel profile) in order to connect the opposite corners of a frame (Figure 3).

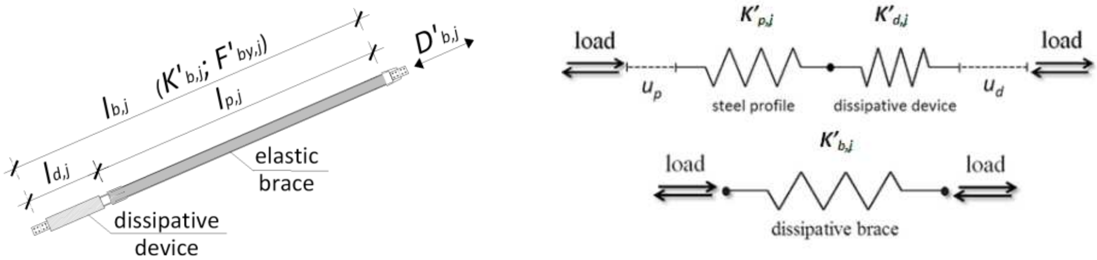

Figure 3 Dissipative device “j” assembled in series with an extension element (e.g. a steel profile): equivalent model of springs in series ($K'_{d,j}$; $K'_{p,j}$) and equivalent single spring model ($K'_{b,j}$)

Therefore, being K'$_{b,j}$ and K'$_{by,j}$ the equivalent stiffness of the spring series in the elastic and plastic range respectively, a= K'$_{p,j}$/K'$_{d,j}$ the ratio between elastic stiffness of the steel profile and of the device and $\beta_{d,j}$ the ratio between stiffness after and before yielding of the dissipative device, the following expression can be derived:

$$K'_{b,j} = \frac{K'_{d,j}}{\frac{1}{\alpha_j}+1} \; ; \qquad K'_{by,j} = \frac{\beta_{b,j} K'_{d,j}}{\frac{\beta_{b,j}}{\alpha_j}+1} \; ; \qquad \alpha_j = \frac{K'_{p,j}}{K'_{d,j}} \tag{15}$$

Therefor:

$$F'_{b,j} = F'_{by,j} + (\delta'_j - \delta'_{y,j}) \frac{\beta_{b,j} K'_{d,j}}{\frac{\beta_{b,j}}{\alpha_j}+1} \tag{16}$$

$$\delta'_{y,j} = \frac{F'_{by,j}}{K'_{b,j}} = \frac{F'_{by,j}}{K'_{d,j}} \left(\frac{1}{\alpha_j}+1\right) \tag{17}$$

Consequently, if there is one brace per direction and per floor, substituting Eq. (16) into Eq. (14), $v^*_{eq,B}(D^*_t)$ can be expressed in the following way:

$$v^*_{eq,B}(D^*_t) = \chi_B \frac{2}{\pi} \frac{\sum_{j=1}^{n} \left\{ F'_{by,j}\delta'_j - \delta'_{y,j} \cdot \left[F'_{by,j} + (\delta'_j - \delta'_{y,j}) \frac{\beta_{d,j} K'_{d,j}}{\frac{\beta_{d,j}}{\alpha_j}+1} \right] \right\}}{F_{S,S+B}(D^*_t) \cdot D^*_{S,S+B}} \tag{18}$$

δ'_j is determined from the pushover analysis for the top displacement D_t and $\delta'_{y,j}$, that is the yielding displacement of devices, can be reasonable assumed as $\delta'_{y,j} \leq \delta'_j / 4$.

$F'_{y,j}$ is, for each direction, the yielding force of the floor brace: once $\delta'_{y,j}$ has been defined $F'_{y,j}$ is consequently determined Eq. (17). Thus, according to Eq. (15), $K'_{d,j}$ can be expressed as follows:

$$K'_{d,j} = K_{global} \cdot c_{b,j} \cdot \left(\frac{1}{\alpha_j}+1\right) \tag{19}$$

Therefore substituting Eq. (19) into Eq. (18), K_{global} can be determined as follows:

$$K_{global} = \frac{\pi \cdot v^*_{eq,B}(D^*_t) \cdot F_{S,S+B}(D^*_t) \cdot D^*_{S,S+B}}{2 \cdot \chi_B \cdot C_1} \tag{20}$$

With:

$$C_1 = \sum_{j=1}^{n} c_{b,j} \left\{ \delta'_{y,j} \cdot \delta'_j - \delta'_{y,j} \left[\delta'_{y,j} + (\delta'_j - \delta'_{y,j}) \frac{\beta_{b,j}\left(\frac{1}{\alpha_j}+1\right)}{\frac{\beta_{b,j}}{\alpha_j}+1} \right] \right\} \tag{21}$$

A value of $\alpha_j > 3$ is usual in applications, therefore K'$_{b,j}$>3/4K'$_{d,j}$, while the steel profile must be stronger (neither yielding nor buckling) than the device: for a given interstorey drift the larger is α_j the larger are device displacements and hysteretic cycles. At this point all terms of Eq. (20) are known so, from Eq. (19) and Eq. (15) , the floor brace stiffnesses $K'_{b,j}$ can be defined (the yielding force $F'_{by,j}$ can be directly derived since the stiffness $K'_{b,j}$ and the yielding displacement $\delta'_{y,j}$ have been defined). Though in this paper the procedure is discussed referring to Eq. (18) it is important to underline that, in a general case, one can have m different braces for each level j. In fact, at the same level, each brace i can be characterized by its specific properties as a consequence, for example, of the geometry of the bays of the structural frame. Consequently Eq. (18) can be generalized as follows.

$$v^{*}_{eq,B}(D_t^{*}) = \frac{2}{\pi}\,\frac{\sum_{j=1}^{n}\sum_{i=1}^{m}\chi_{B,i}\left\{F'_{by,j,i}\delta'_j - \delta'_{y,j,i}\cdot\left[F'_{by,j,i} + (\delta'_j - \delta'_{y,j,i})\dfrac{\beta_{d,j,i}K'_{d,j,i}}{\dfrac{\beta_{d,j,i}}{\alpha_{j,i}}+1}\right]\right\}}{F_{S,S+B}(D_t^{*})\cdot D^{*}_{S,S+B}} \quad (22)$$

(10) Check convergence: one must repeat steps from 3 to 9 until the performance point of the braced structure converges to the target displacement with adequate accuracy.

APPLICATION OF THE DESIGN PROCEDUREC TO A REGULAR R.C. FRAME

The proposed design procedure has been applied to retrofit an existing r.c. frame structure (Figure 4) designed to resist vertical loads only. The procedure has been applied considering the structure both bare, as in the common professional practice, and infilled in order to design a bracing system able to prevent damages or dangerous collapse of the infill walls (Figure 5 and 6).
Nonlinear dynamic analyses have been performed to assess the effectiveness of the proposed procedure (Figure 7).

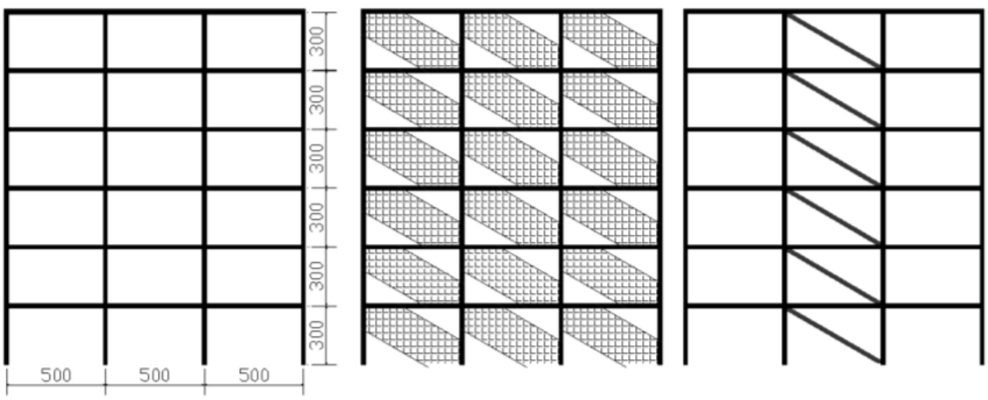

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Figure 4 2D r.c. frames selected as case study [cm]: (a)3x6 r.c. bare frame,(b) 3x6 r.c. infilled frame (infills replaced by the equivalent single strut);(c) distribution of *BRBs* adopted

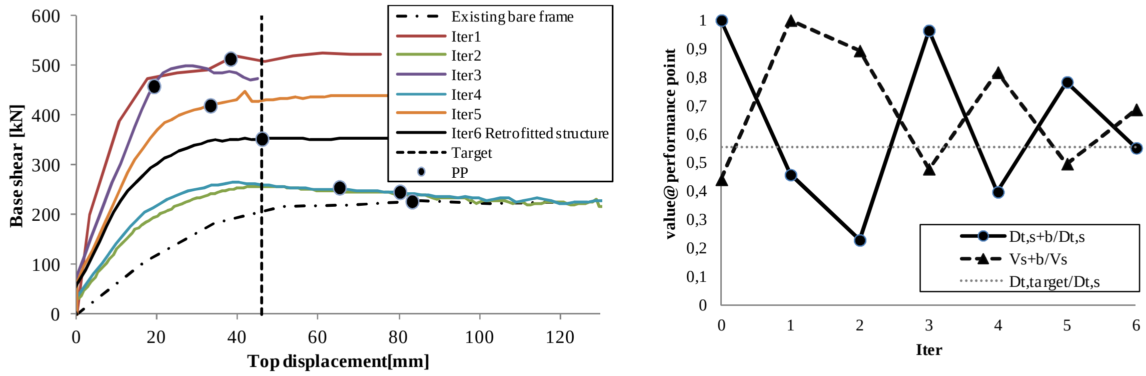

Figure 5 Bare frame. Variation of response with iterations of the procedure and target displacement $D_{t,target}$. Capacity curve and relative performance point at each iter (left); top displacement $D_{t,s+b}$ and base shear V_{s+b} at performance point of each iter normalized with respect to the maximum corresponding value (right)

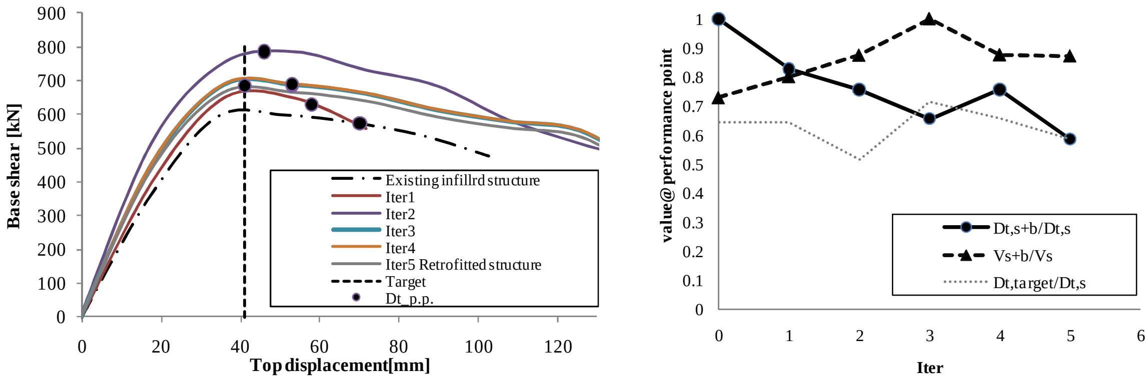

Figure 6 Infilled frame. Variation of response with iterations of the procedure and target displacement $D_{t,target}$. Capacity curve and relative performance point at each iter (left); top displacement $D_{t,s+b}$ and base shear V_{s+b} at performance point of each iter normalized with respect to the maximum corresponding value (right)

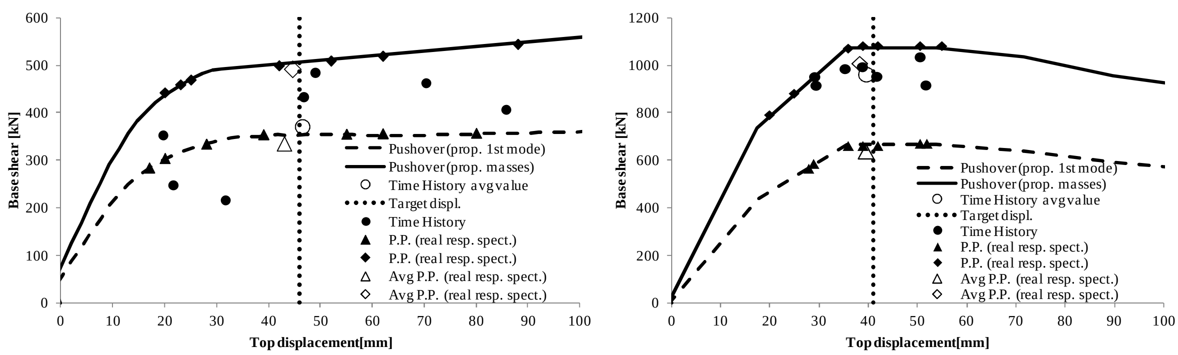

Figure 7 Comparison between pushover (load distribution prop. to masses and to 1^{st} mode), incremental nonlinear dynamic analyses (response to each T.H. and their average value) and performance point determined using the response spectrum of each T.H. of the retrofitted structures: bare frame with bracing (left), infilled frame with bracing (right)

APPLICATION OF THE DESIGN PROCEDUREC TO AN IRREGULAR BUILDING

It is well known that results from a non linear static analysis are influenced by: pushover loading profile, characteristics of the numerical models. The loading profile determines loads distribution and deformed shape of the building and, consequently, the plastic distribution of forces and displacements (interstorey drift can be strongly influenced). The most common loading profiles are: proportional to masses, proportional to first mode shape (monomodal), proportional to acceleration, multimodal. The procedure presented in the previous chapter is generally applied using a “standard” monomodal pushover where the structure is subjected to monotonically increasing lateral forces, with an invariant spatial distribution (fundamental mode based), until collapse displacement is reached. This fundamental mode based force distribution doesn’t account for higher mode contribution, which can be relevant, and therefore this limits the applicability of this approach to cases where the fundamental mode is dominant. Anyway it has to be highlighted that braces, if well designed, regularize the structure that can become strongly fundamental mode dependent. As discussed in the following has been analyzed, with a specific case study, if the use of the simple monomodal approach can be considered efficient. Therefore the proposed design procedure has been also applied to retrofit an existing r.c. frame structure (Fig. 8-9) designed to resist vertical loads only: it is a strategic building, situated in a seismic area of Italy, that has been designed and built in the 1970s without seismic details.

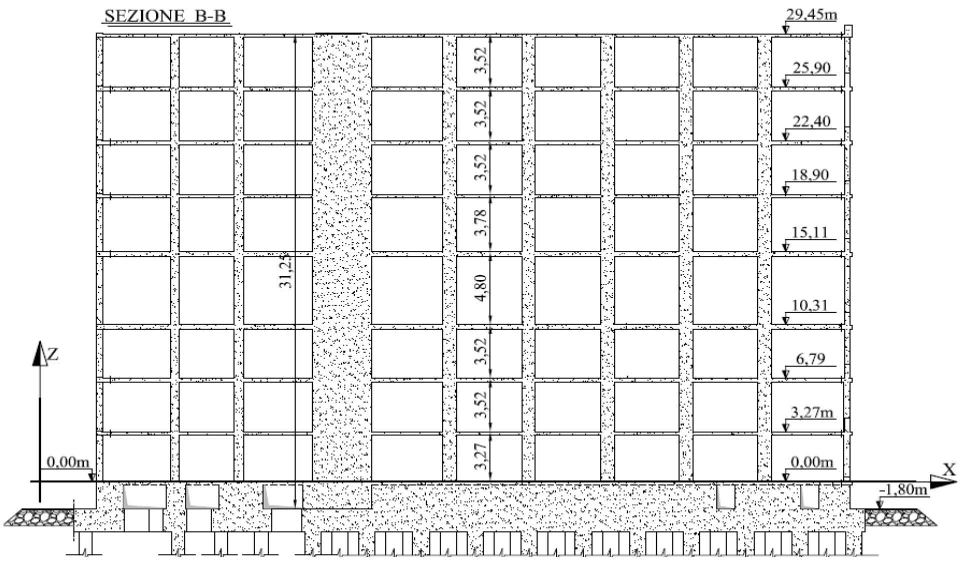

Figure 8: Longitudinal sections of the building

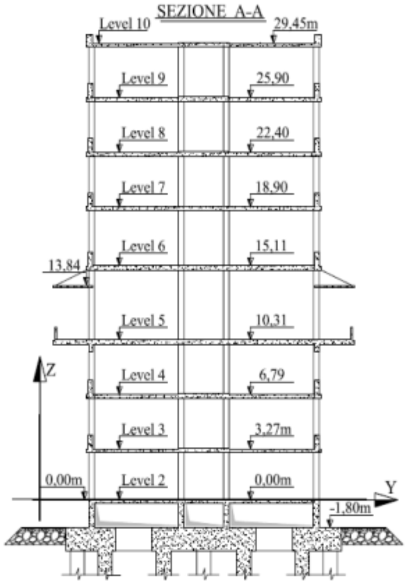

Figure 9: Transverse section of the building

For brevity, in this paper, only results from longitudinal analysis are described: the design process has been performed considering the real 3D structure. The procedure has been applied in order to retrofit the building referring to a seismic action evaluated using the technical code currently in force in Italy (*p.g.a.* 0.25g; return period 949 years).

The capacity curve has been derived considering a loading profile proportional to the first mode shape. In addition both the existing structure and the retrofitted structure have been studied using the multimodal pushover (Chopra & Goel, 2002) in order to evaluate the effectiveness of the ”standard” procedure and therefore the advantages on using the monomodal pushover for such a building: in this case the effectiveness of the procedure has been confirmed and, comparing results from monomodal and multimodal pushover applied on the retrofitted structure, the use of multimodal pushover can be considered not substantial for the design process applied on this typology of building. The performance point of the existing structure in terms of base shear and top displacement is V_S =9908 kN and $D_{t,S}$=133 mm. Then, the selected performance objective was to reduce displacement in order to avoid damage on both r.c. elements and masonry panels.

Therefore the target displacement has been selected adopting the following parameters: reducing the top displacement of about 50% ($D_{t,S,targ}$=66 mm) and limiting the interstorey drift to 2‰ at whichever level. Convergence to the desired values has been obtained with three iterations and the final result (performance point, iter 3) is the base shear V_{S+B}=12105 kN, with a 19% increase with respect to the original building, and the top displacement $D_{t,S+B}$=61 mm (practically coincident with the target, see Fig. 10). The contribution to dissipation offered by the dissipative system is ν_{eqB}=20% (ν_{eqS}=12%, ν_I=5%). In the final configuration the interstorey drift of each level has been significantly reduced to values lower than 2‰ and all the dissipative braces are in their plastic range (Fig. 12).

The braced structure, if the distribution of interstorey drift is analyzed, is strongly characterized by a dominat first mode and consequently the multimodal analysis can be considered unnecessary: the two approaches differ of less than 1.5% (Fig. 13).

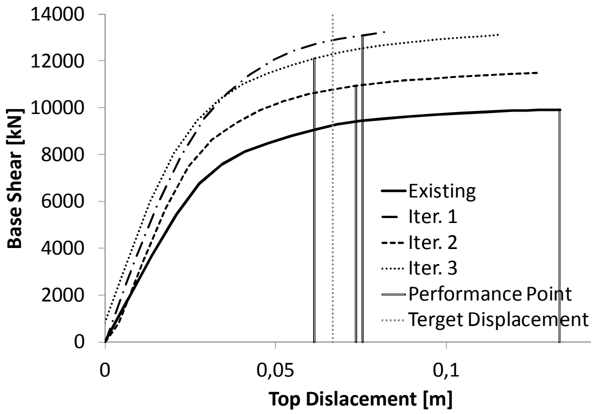

Figure 10: Capacity curves from pushover analysis along the longitudinal direction (Existing structure and braced structure at each iter from 1 to 3)

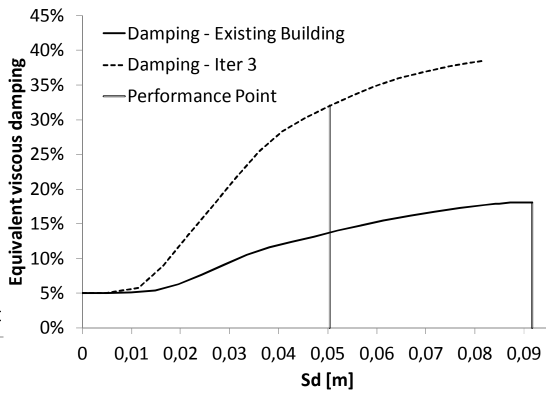

Figure 11: Variation of the total equivalent viscous damping with the spectral displacement S_d. At the p.p. $\nu_{eq,S+B}$=32% for the retrofitted structure and $\nu_{eq,S}$=14% for the existing building.

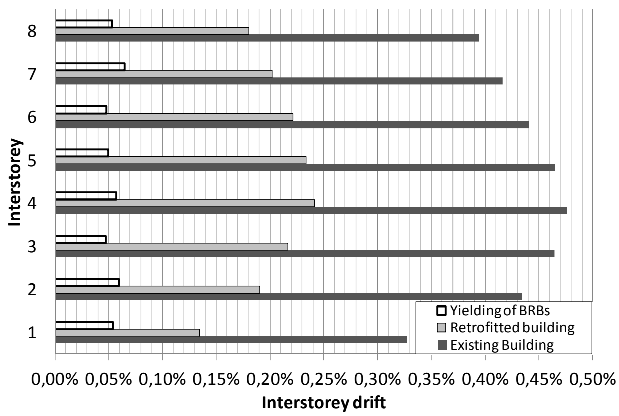

Figure 12: Interstorey drift (longitudinal) distribution for the existing building and the retrofitted building (longitudinal direction). In the graph is also indicated the drift corresponding to the yielding of the BRBs

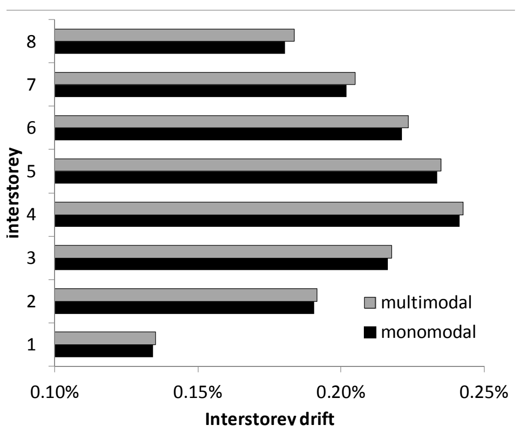

Figure 13: Interstorey drift (longitudinal) distribution in the retrofitted building obtained using the monomodal and the multimodal pushover. Differences are lower than 1,5%

CONCLUSIONS

A procedure for design of retrofitting of building structures using dissipative braces has been presented. The procedure, has been applied to retrofit a regular frame building according to the method that has been described in previous publications (*Bergami & Nuti*, *2013*). Moreover it has been applied to an irregular structure (with shear walls and frames) having eight floors (30 m height): for this application the standard procedure has been updated with the multimodal pushover. For both the case study the target displacement has been determined, both in longitudinal and transverse direction, in order to limit both interstory drifts and ductility demand on existing structural elements. For the first case study non linear dynamic analyses have been performed to assess the effectiveness of the proposed procedure and define with which load distribution the pushover analysis should be performed if considering or not the presence of the infills: in case of a masonry infilled frame the constant load distributions seems to be the best one instead for the bare frame a distribution proportional to te first modal shapes gives best results. Concerning the second case study the final configuration obtained (the building braced along both the directions) has been tested performing pushover analyses proportional to the most relevant mode shapes (along both the longitudinal and transverse direction) of the building fully braced and, afterwards, results have been compared with results from multimodal pushover. From this comparison has been observed that, in terms of drifts and displacements, the multimodal pushover can be considered not relevant for the design procedure for dissipative braces proposed by *Bergami & Nuti* (2013) if the configuration of the braces regularize the building. Of course in many cases the use of multimodal pushover is suggested: if the designer is not free to place the braces in an ideal configuration or if the structure is strongly irregular and the designer wants to reduce the iterations number.

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