ASEISMIC RETROFITTING OF REINFORCED CONCRETE FRAME AND CONCENTRIC BRACED STEEL BUILDINGS WITH DISSIPATIVE BRACINGS

(ADEGUAMENTO SISMICO MEDIANTE CONTROVENTI DISSIPATIVI DI STRUTTURE A TELAIO IN C.A. E TELAI IN ACCIAIO CON CONTROVENTI CONCENTRICI)

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Parole chiave: controventi dissipativi, adeguamento sismico.

Sommario. In questo articolo viene presentato un approccio progettuale per sistemi di controventi dissipativi per l'adeguamento sismico di edifici esistenti già presentato nel *2nd International Conference on Steel and Composite Structures for Large-scale Buildings* (Shanghai, Aprile 2014). La procedura progettuale proposta deriva da un approccio progettuale ideato per l'adeguamento sismico di telai in c.a. (*Bergami A.V., Nuti C., 2013*) e che viene qui discusso anche per l'uso su telai in acciaio con controventi concentrici (CBF). Il metodo può essere adottato per la progettazione di sistemi composti da qualsiasi tipologia di controvento dissipativo esistente sul mercato e può essere calibrato caso per caso, in base alle specifiche esigenze del progettista, col fine di raggiungere la performance strutturale desiderata in termini sia di spostamento massimo sia di configurazione deformata. L'approccio è di semplice applicazione poiché si basa su analisi di tipo statico non lineare (pushover), evitando quindi il ricorso ad onerose e complesse analisi dinamiche: tale caratteristica rende l'approccio proposto particolarmente adatto all'uso professionale. All'interno della procedura, col fine di renderne quanto più ampio possibile il campo di applicabilità, l'analisi statica non lineare può essere condotta sia adottando forme di distribuzione delle forze statiche incrementali di tipo convenzionale (monomodale, proporzionale alle masse di piano, ecc.) sia di tipo multimodale (*Chopra, A.K., Goel R.K., 2002)*: l'approccio multimodale consente di migliorare l'attendibilità del risultato ottenuto anche in caso di strutture sensibili ai modi superiori per elevato numero di piani o irregolarità. Nell'articolo viene discussa sia l'applicazione ai CBF sia un caso studio di una struttura esistente irregolare a telaio in c.a. tamponato ove l'uso di pushover sia tradizionale che multimodale viene analizzato.

1 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. 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. As discussed in Bergami & Nuti (2013), the design of dissipative devices have two main goals: improve dissipation and regularize strength end stiffness distribution (this can be done adopting an adequate criteria to distribute the braces along the elevation and inside the plan of the building).

Moreover, in case of medium rise building (quite widespread in Italy), it is a matter of fact that the relevance of higher modes depends not only on their level of irregularity but it is also related to the quite high number of stories. To check such hypothesis the design procedure, that is detailed in is general form and specialized to the case of steel braced frames, has been tested on a medium rise irregular existing r.c. building. The necessity of using a multi modal pushover (Goel & Chopra, 2004) instead of the standard single mode pushover procedure has been investigated performing multimodal pushover and non linear dynamic analysis on both the existing and retrofitted building.

2 DISSIPATIVE BRACINGS POSITIONING: 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, induce 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 profile) 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.

3 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: $E_{D,S}$ 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

$F_s(D)$ 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 χ_S for the structure and χ_B for the braces (χ =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 Buckling-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 $\nu_{eq,S+B}$, to be added to the inherent damping ν_I (usually ν_I =5% for r.c. structures and ν_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 $\nu_{eq,B}$ 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 $S+B$.

4 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-S_a).
2. **Select the target displacement**: the target displacement is selected (for example the top displacement D_t^*) 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 (D_t-V_b), 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 D_t-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).

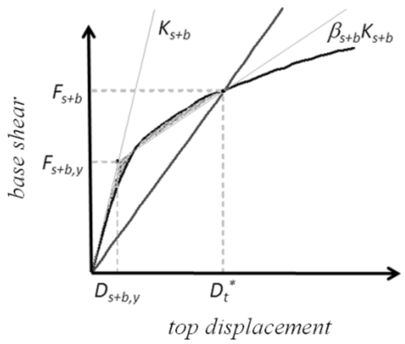

Figure 1: Evaluation of the equivalent bilinear capacity curve

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 ($\Gamma=(\phi^T MI)/(\phi^T M\phi)$) and $L=\phi^T MI$.
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 h 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}^*.

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

7. **Evaluate the equivalent viscous damping contribution due to the naked structure**: the contribute to damping of the structure $\nu^*_{eq,S}(D^*_t)$ can be determined from Eq. (3.12) being D_t^* the top displacement corresponding to $E^{bilinear}_{D,S}$ and $E_{S,S+B}$ that are the energy dissipated by S and the elastic strain energy of $S+B$ ($E^{bilinear}_{D,S}$ 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 ν^*_{tot} from Eq. (12) the equivalent viscous damping needed to be supplied by the braces $\nu^*_{eq,B}(D^*_t)$ is evaluated from Eq. (3.11) and Eq. (4.1) as follows:

$$\nu^*_{eq,B}(D^*_t) = \nu^*_{tot}(D^*_t) - \nu^*_{eq,S}(D^*_t) - \nu_I \tag{13}$$

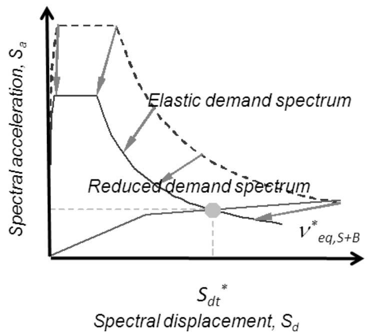

Figure 2: Evaluation of the equivalent viscous damping needed to achieve the target performance point

9. **Dimensioning of the braces**: once the required equivalent viscous damping $\nu^*_{eq,B}(D^*_t)$ 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 j_{th} level can be expressed as:

$$E^{bilinear}_{D,B} = \sum_{j=1}^{n} 4\left(F'_{by}\delta'_j - \delta'_{y,j}F'_{b,j}(\delta'_j)\right) \tag{14}$$

being δ'_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 δ'_{bj} 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*) assembled in series with an extension element (*e.g.* realized with a steel profile) in order to connect the opposite corners of a frame (Fig. 3).

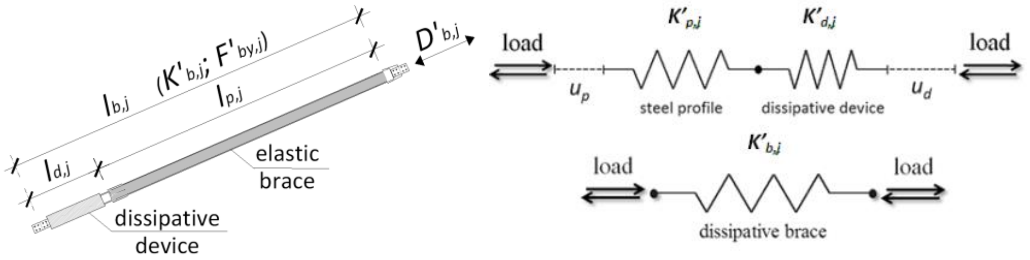

Figure 2: Dissipative device “j” assembled in series with an extension element (e.g. a steel profile): equivalent model of springs in series ($K'_{d,j}$; $K'_{p,j}$) and equivalent single spring model ($K'_{b,j}$)

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:

$$K_{b,j}^{'} = \frac{K_{d,j}^{'}}{\dfrac{1}{\alpha_j}+1}\;;\;\; K_{by,j}^{'} = \frac{\beta_{b,j}K_{d,j}^{'}}{\dfrac{\beta_{b,j}}{\alpha_j}+1}\;;\;\; \alpha_j = \frac{K_{p,j}^{'}}{K_{d,j}^{'}} \tag{15}$$

Therefore:

$$F_{b,j}^{'} = F_{by,j}^{'} + (\delta_j^{'} - \delta_{y,j}^{'})\frac{\beta_{b,j}K_{d,j}^{'}}{\dfrac{\beta_{b,j}}{\alpha_j}+1} \tag{16}$$

$$\delta_{y,j}^{'} = \frac{F_{by,j}^{'}}{K_{b,j}^{'}} = \frac{F_{by,j}^{'}}{K_{d,j}^{'}}\left(\frac{1}{\alpha_j}+1\right) \tag{17}$$

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:

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

δ'_j are 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 $\delta_{y,j}^{'} \le \delta_j^{'}/4$.

$F_{y,j}^{'}$ 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:

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

A value of $a_j>3$ 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 a_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.

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

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.

4.1 Application to steel concentric braced frames (CBF)

Recent studies and experimental courses have pointed out that the seismic capacity of steel concentric braced frames (CBF) is limited by diagonal yielding while buckling of compressed diagonals can be tolerated if they remain in the elastic range. The contribution of compressed diagonals is disregarded in design, though it can affect seismic response, reducing the period of vibration of the structure, while increasing the absorbed energy.

The retrofit procedure previously presented has been specialized also to the case of CBFs, and is presented in this paragraph. The structural elements: beams and columns usually act as pendulum, hinged at their ends, though this assumption can be removed without any modification to the presented procedure.

The procedure allows to design a dissipative bracing system that can guarantee the upgrade of the existing CBF avoiding yielding in tension of the existing diagonals, or even, in some cases, to prevent compressed original diagonal from elastic buckling. In the latter case the pushover analysis can be easily performed using a software for structural analysis in fact, this typologies of structures (CBF) after the retrofitting should remain linear elastic even under seismic action (in some cases, as a rare seismic event, a limited plastic excursion of the diagonals could be accepted to guarantee to survive the event) and therefore the nonlinearity can be concentrated only in the dissipative devices. In case one accept elastic buckling of the compressed original braces, one should consider the nonlinear elastic behavior of these elements, unless it is proved that their contribution to total stiffness is negligible, and disregarded in the analysis.
The specialized procedure can be summarized by modifying steps 2 and 3 as follows:
2. **Select the target displacement**: the target displacement is selected (for example the top dis-

placement D_t^*) according to the performance desired (limit state). Usually, working with steel buildings, deformation limits are relevant for the dimensioning. For CBF (concentric braced frame) the target can be selected as the inter story drift that corresponds to the stability limit of the existing braces in compression. When this latter is too small to be considered than one should accept elastic buckling and displacement corresponding to yielding in tension of the existing diagonals should be the maximum admissible state at least for life safety. Larger target displacement are usually of small practical interest.

3. **Define the capacity curve**:. Given the target CBF displacement, the analysis can be reduced to the evaluation of the top displacement that corresponds to the achievement yielding in tensile original bracings or at most, the prevention of buckling in compressed original braces. This latter seems an excessive request as buckling in the elastic range has small consequences on structural damage and should usually be accepted. However if one prevent buckling than the capacity curve of the existing CBF (that is the S of the procedure) will be linear elastic, otherwise it will be non linear elastic due to compressed buckled braces. When their contribution to structural stiffnes is negligible, less than 15%, they can certainly be disregarded.

If we disregard compressed diagonals or assume the stability limit of compressed diagonals as performance point, we shall design an additional bracing system that yields well before the performance point (reduction of the interstorey drift to the critical deformation that corresponds to diagonal yielding or buckling: ultimate limite state that corresponds to the ultimate top displacement *Du*). The retrofitted system *S+B* would be elasto-plastic with a post yielding hardening (elastic before reaching $D_{b+s,y}$ and hardening in $D_{b,y}$-D_u). The CBF (S) is in its elastic range (Figure 4), and the structure yields when the dissipative braces yields $D_{b,y}=D_{b+s,y}$.

In case we admit elastic buckling in compressed original braces and we cannot disregard their contribution to structural stiffness, than the pushover curve of the original struscture: S will be non linear elastic, we shall add the elasto plastic curve of the new braces to the original curve. In this case one should note that for the same displacement, the original structure has a larger response with respect to the case of disregarded compressed diagonal, and the equivalent damping is smaller adding the same BRB due to the larger elastic energy at the denominator of expr. 9).

During the design phase it is important to evaluate if, inside the range of displacement that allow to dissipate enough energy to obtain the performance target required, the existing diagonals remain elastic (*e.g.* according to code requirements). If this condition is verified the dissipative braces will be inserted inside the structure without modifying the CBF configuration.

In the opposite case the designer can decide, for example, to remove the existing diagonals and realize a completely new system of diagonals with the dissipative braces.

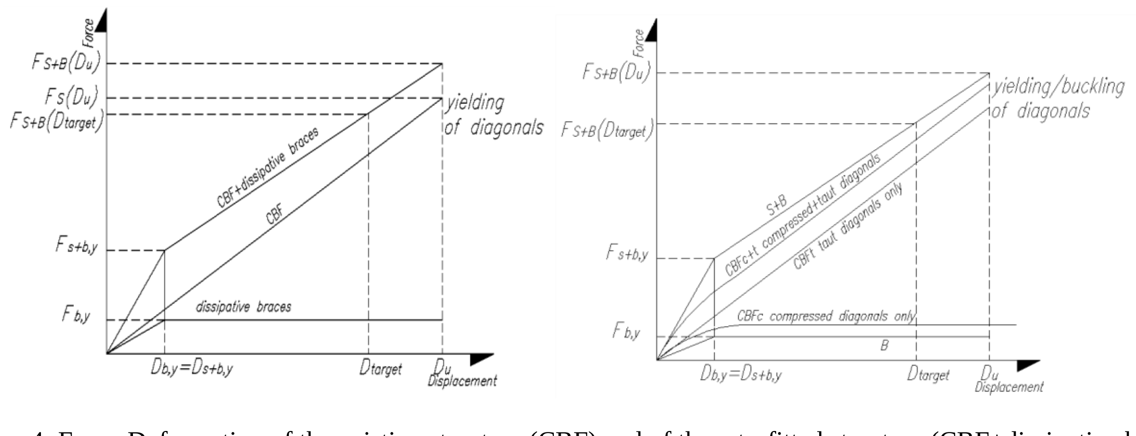

Figure 4: Force-Deformation of the existing structure (CBF) and of the retrofitted structure (CBF+dissipative braces). D_{target} is the target of the retrofitting design: diagonals must remain elastic and $D_{target}<D_u$.
Left: original compressed braces neglected. Right: original compressed braces considered. B is the bracing systems (the BRBs) and S is the existing structure (CBF=CBFc+CBFt).

In the following Figures the deformed shape before and after retrofitting are illustrated.

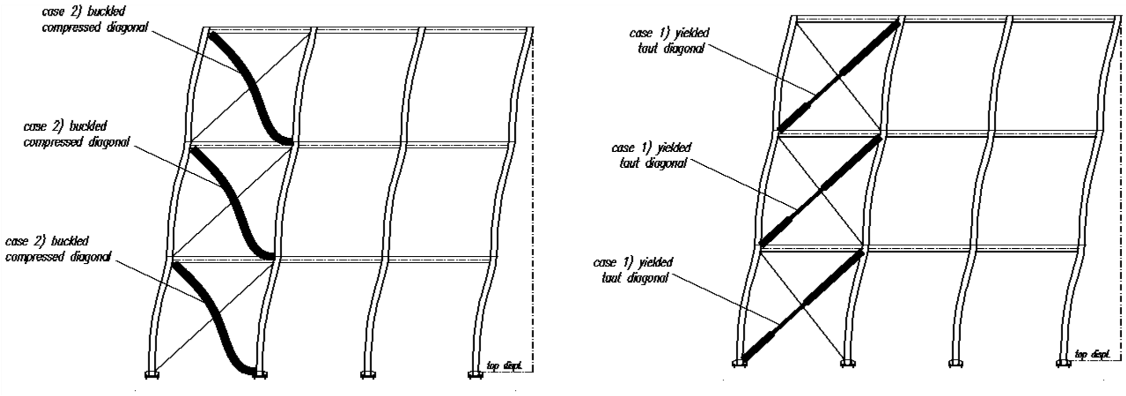

Figure 5: existing structure (CBF) under seismic action. The taut diagonals yielded (case 1 on the right), the compressed diagonals buckled (case 2 on the left) when the top displacement $D_{top}=D_u$

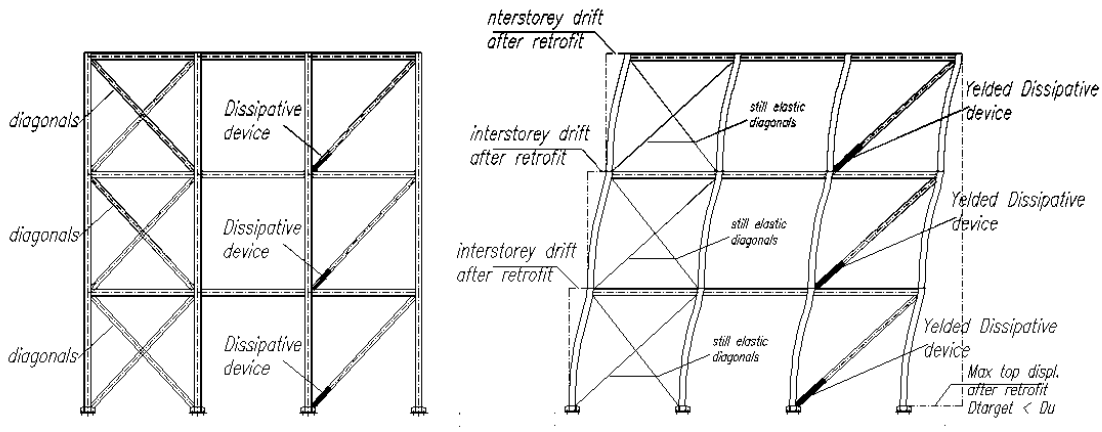

Figure 6: Retrofitted structure (CBF+dissipative braces) before and during seismic action. The diagonals are still elastic and the dissipative braces are yielded D_{top} after retrotting is limited in order to obtain the retrofitting of the structure: $D_{top}=D_{target}>D_u$

5 APPLICATION ON AN EXISTING R.C. STRATEGIC 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 applied to retrofit an existing r.c. frame structure (Fig. 7-8) 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 1970*s* without seismic details.

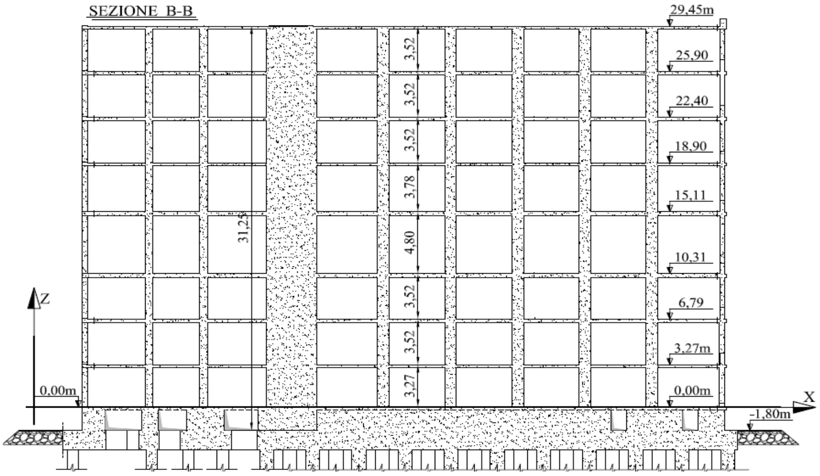

Figure 7: Longitudinal sections of the building

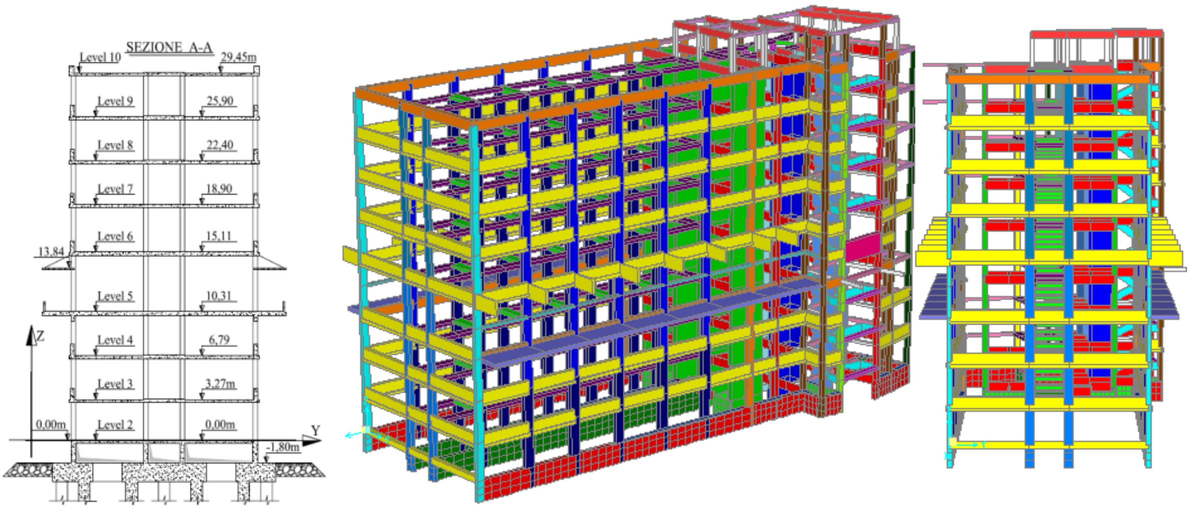

Figure 8: Transverse section of the building and 3D view of the numerical model

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. 9).

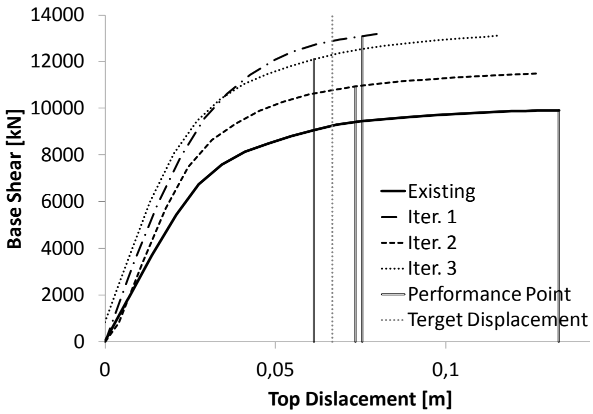

Figure 9: Capacity curves from pushover analysis along the longitudinal direction (Existing structure and braced structure at each iter from 1 to 3)

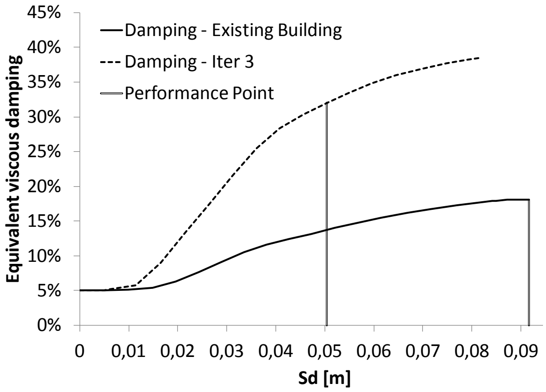

Figure 10: Variation of the total equivalent viscous damping with the spectral displacement S_d. At the p.p. $\nu_{eq,S+B}$=32% for the retrofitted structure and $\nu_{eq,S}$=14% for the existing building.

The contribution to dissipation offered by the dissipative system is ν_{eqB}=20% (ν_{eqS}=12%, ν_I=5%): Figure 10. 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. 11). 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. 12).

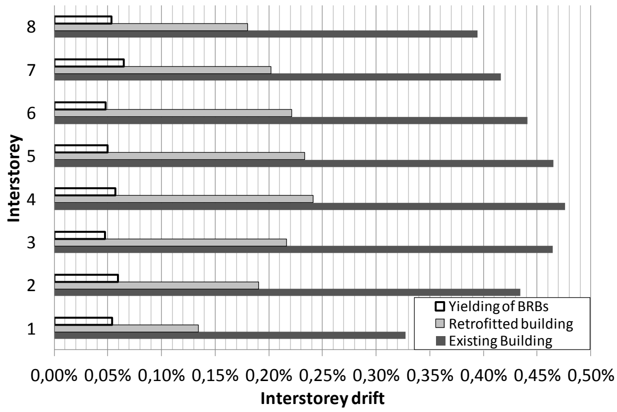

Figure 11: Interstorey drift (longitudinal) distribution for the existing building and the retrofitted building (longitudinal direction). In the graph is also indicated the drift corresponding to the yielding of the BRBs

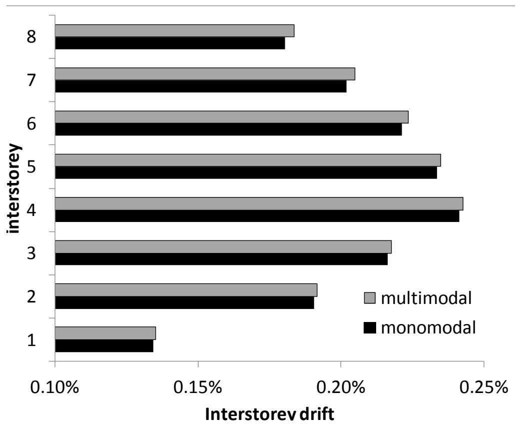

Figure 12: Interstorey drift (longitudinal) distribution in the retrofitted building obtained using the monomodal and the multimodal pushover. Differences are lower than 1,5%

6 CONCLUSIONS

A procedure for design of retrofitting of building structures using dissipative braces has been presented and the specific case of r.c. frame structures and CBFs have been detailed. The procedure, that in previous works has been applied to retrofit building up to four stories (Bergami & Nuti, 2012), has been updated with the use of the multimodal pushover and successfully applied to a structure having eight floors (30 m height). 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. 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. Afterwards, results obtained from the application of monomodal pushover have been compared with results from multimodal pushover: the effectiveness of the procedure has been proved. Moreover, 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 and Nuti (2012) if it is applied on structures such as the one analyzed (midrise r.c. frame building).

REFERENCES

[1] Applied Technology Council. (1996). Seismic evaluation and retrofit of concrete buildings. Report ATC-40, Redwood City, California.

[2] Bergami A.V., Nuti C. (2013). A design procedure of dissipative braces for seismic upgrating structures. *Earthquakes and Structures,* Vol. **4**, No. 1, 85-108.

[3] Bergami A.V., Nuti C. (2011). Seismic retrofit of r.c. structures with dissipative braces, design and sustainability. fib Symposium 2011, Prague, Czech Republic.

[4] FEMA-274, (1997). “NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings”. Federal Emergency Management Agency Publication, U.S.A., 274.

[5] FEMA – ASCE 356 (2000). “Prestandard and Commentary for the Seismic Rehabilitation of Buildings”. Washington, DC, 2000.

[6] Chopra, A. K., and Goel, R. K., 2002. Modal pushover analysis procedure for estimating seismic demands for buildings, Earthquake Eng. Struct. Dyn. 31 (3), 561–582.

[7] Black RG, Wenger WAB, Popov EP. Inelastic buckling of steel struts under cyclic load reversals. In: Rep. No. UCB/EERC-80/40. Berkeley, California; 1980.

[8] Goel SC. Earthquake resistant design of ductile braced steel structures. In: Stability and ductility of steel structures under cyclic loading. 1992; CRC Press. p. 297–308.

[9] Tremblay R. Inelastic seismic response of steel bracing members. J Constr Steel Res 2002;58:665–701.

[10] Bergami A, Liu X, Nuti C, Zhou Z. Seismic retrofitting of reinforced concrete frame and concentric braced steel buildings with dissipative bracings. *The 2nd International Conference on Steel and Composite Structures for Large-scale Buildings*. Shanghai, China, 17-19 April, 2014