Energy-based Design Method for Passive Energy Dissipative Bracing Systems

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Energy-based design method
for passive energy-dissipative bracing systems

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Abstract
In this study an energy-based method for the design of passive Energy Dissipative Bracing (EDB) systems is presented, as a retrofit technique for existing reinforced concrete (RC) buildings. A comprehensive literature overview concerning the design of hysteretic bracing systems based on various design philosophies, such as force-, displacement- or energy-based, is provided. The efficiency of the proposed method is validated by comparing the proposed methodology with two design procedures selected in the literature, applied to three RC frames. The results showed that the proposed method is more effective in avoiding the damage concentration at a single story and in distributing the additional strength provided by the EDBs proportionally to the hysteretic energy demand along the structure height. The validity of each procedure is compared based on non-linear static and non-linear dynamic analyses.

1 Introduction
Seismic design methods are either force-, displacement- or energy-based. It is widely accepted that force-based approach is not a suitable tool for implementing performance-based earthquake engineering concept (Bertero and Bertero 2002). Performance levels are more effectively described in terms of displacements, as damage is better correlated to displacements rather than forces. Therefore, efforts have been made to develop alternative methods based on displacement- and energy-based concepts rather than on forces. In this respect, significant research has been carried out for developing design methods of hysteretic bracing systems, based on displacement- and energy-based concepts, to be used in the seismic retrofitting of existing reinforced concrete (RC) buildings.

The main limitations encountered in displacement-based approaches for designing energy-dissipative bracing (EDB) systems are: a) conversion of a MDOF system into a SDOF system and choice of the SDOF displacement as the main design parameter; b) distribution of strength and stiffness along the storeys, accounting for the existing structure characteristics; c) neglecting the cyclic behaviour of structural elements in design; d) not considering the effects of duration-related cumulative damage; e) not accounting for near-fault effects.

These limitations are overcome in energy-based approaches, thus making them more attractive. Since Housner (Housner 1956) proposed the energy balance concept in 1956, significant amount of research has been dedicated to developing design methods based on such concept. For example, Decanini and Mollaioli (Decanini and Mollaioli 2001) proposed two inelastic energy spectra, namely the input energy \( E_I \) and the hysteretic-to-input energy ratio \( E_H/E_I \). These spectra allow to evaluate the seismic demand in terms of maximum displacement and ductility. They further studied the influence of the inelastic behaviour on the input energy spectra and on the excitation characteristic, which is influenced by soil type, source-to-site distance, and seismic event magnitude. Riddel and Garcia
(Riddell and Garcia 2001) presented a hysteretic-energy demand spectrum. Chou et al. (Chou and Uang 2000) proposed an attenuation relationship for establishing absorbed (strain) energy spectra, based on two-stage nonlinear regression analysis on 273 ground motion records. Akiyama (Akiyama 1985), using Japanese design earthquakes, introduced the input energy per unit mass of an elastic SDOF structure due to a given earthquake and provided a relationship between normalized input and damage (hysteretic) energy. Akbas et al. (Shen and Akbas 1999) proposed a linear relationship for the hysteretic energy distribution along the building height, based on several non-linear time history analysis of regular frames with a 2% damping ratio. Ye et al. (Ye et al. 2012) established a relationship between the peak story responses and plastic deformation energy obtained from a pushover analysis. Uang and Bertero (Uang and Bertero 1990) obtained the distribution of hysteretic energy in multi-story buildings.

Several energy-based procedures for the design of energy-dissipative bracing systems have been proposed in the literature: Choi and Kim (Choi and Kim 2006) proposed a design method using hysteretic and accumulated energy spectra and obtained the bracings cross-section by equating the hysteretic energy demand to the accumulated plastic energy dissipated by the braces. Benavent-Climent (Benavent-Climent 2011) proposed a design procedure of EDB systems grounded on the concept of energy-balance, imposing that the whole hysteretic energy demand is dissipated entirely by the EDB system while the existing structure remains elastic. Dasgupta et al. (Dasgupta et al. 2004) obtained the base shear of the EDB system using energy balance and compared it with the base shear obtained from displacement-based design.

Although the efficiency of energy-based procedures is praised in the literature, there are several factors that discourage the practicing engineers to use them: a) the definition of actions in terms of energy and not in terms of the more familiar acceleration response spectrum; b) evaluating the energy demand along the building height; c) inclusion of several parameters in energy-based procedures to account for various earthquake effects, such as number of plastic cycles, earthquake duration, cyclic behaviour of elements, etc. However, the recent vast literature provides enough tools to simplify the application of energy-based procedures: as an example, several proposals are now available for defining both energy spectra and the energy distribution along the building height (Decanini and Mollaioli 2001)(Shen and Akbas 1999). Moreover, regression analyses with several earthquakes have been carried out to study the duration and near-fault and far-field effects of earthquakes and simple parameters have been defined to account for the site-to-fault distance (Manfredi, Polese, and Cosenza 2003).

The main objectives of this study are: 1) analysis of pros and cons of two existing design procedures, which are the most well-founded, in the authors opinion: a) the displacement-based design procedure by Ponzo-Di Cesare (Cesare and Ponzo 2017); b) the energy-based method by Benavent-Climent (Benavent-Climent 2011); 2) proposal of a new energy-based methodology for the design of EDB systems and comparison with the two latter procedures to evaluate its relative efficiency.

The comparison is performed on three 2D frames, whose EDB systems are designed, both, by the two selected procedures and by the proposed method. The post-retrofit performance of the frames is assessed through non-linear static and dynamic analysis.

2 Energy balance

The equation of motion for a single-degree-of-freedom (SDOF) inelastic system subjected to a ground motion is given by:

\[ m \ddot{u} + c \dot{u} + f_s = -m \ddot{u}_g \]  

(1)

where \( m \) = mass, \( c \) = damping coefficient; \( f_s \) = restoring force; \( \ddot{u}_g \) = ground acceleration, and \( \ddot{u}, \dot{u}, u \) are acceleration, velocity, and displacement of the system respectively. Multiplying (1) by \( du = \dot{u} dt \) and integrating it over the entire duration of the earthquake, i.e., from \( t = 0 \) to \( t = t_o \), Eq. (1) becomes:
\[
\int_0^{t_0} m \ddot{u} \, dt + \int_0^{t_0} c \ddot{u}^2 \, dt + \int_0^{t_0} f_\xi \, \dot{u} \, dt = - \int_0^{t_0} m \ddot{u}_g \, \dot{u} \, dt
\]  
(2)

where each term can be written as:

\[
E_k + E_\xi + E_s = E_I
\]  
(3)

where \( E_k \) is kinetic energy, \( E_\xi \) is damping energy, \( E_s \) is strain energy, and \( E_I \) is input energy. The strain energy \( E_s = E_{se} + E_p \) is made of two parts: recoverable elastic strain energy \( E_{se} \), and irrecoverable plastic strain energy \( E_p \).

Eq. (3) can be rewritten as:

\[
E_k + E_\xi + E_{se} + E_p = E_I \implies E_e + E_p = E_H
\]  
(4)

where \( E_e = E_k + E_{se} \) is the elastic strain energy, \( E_p \) is the plastic strain energy and \( E_H = E_I - E_\xi \) is the hysteretic energy demand. This can be found using energy spectra, e.g., those proposed in (Decanini and Mollaioli 2001).

The elastic strain energy \( E_e \) occurs because of the elastic deformation of the structure and becomes null when vibration of the structures ends. The plastic strain energy \( E_p \) is related to the inelastic deformation that the structure undergoes during the ground motion. Unless otherwise dissipated through some mechanism, \( E_p \) generally inflicts permanent damage to the structure. The objective of retrofitting gravity-load-designed structures is to dissipate \( E_p \) through supplemental devices, the EDB system, while the existing structure remains elastic. This implies that, globally:

\[
E_p \geq E_H
\]  
(5)

This is achieved if at each \( i \)-th story the following requirement is fulfilled:

\[
E_{p,i} \geq E_{H,i}
\]  
(6)

The above requirement is satisfied if, at each \( i \)-th storey it can be ensured that:

\[
d_{bu,i} \leq d_{fy,i}
\]  
(7)

where \( d_{bu,i} \) is the ultimate displacement of the EDB system at the \( i \)-th storey, and \( d_{fy,i} \) is the yield displacement of the existing frame at the \( i \)-th storey. That is, the EDB system dissipates energy before the frame yields at any \( i \)-th storey. If this is satisfied, then the global plastic strain energy \( E_p \) is due to the dissipation capacity of the EDB system only, purposely designed to dissipate the global hysteretic energy demand \( E_H \).

### 3 Proposed methodology

#### 3.1 Shear force coefficient

The proposed procedure exploits the strength distribution profile of an optimally designed EDB system proposed by Akiyama (Akiyama 1985), who suggested that buildings with medium height can be represented by a shear strut. By solving the dynamic equation of the shear strut under ground motion, Akiyama arrives at proposing a shear coefficient ratio:

\[
\bar{\alpha}_i = \frac{\alpha_i}{\alpha_1} = \frac{Q_{b,i}}{Q_{b,1}} \Gamma_i
\]  
(8)
where $\alpha_i = \frac{Q_{b,i}}{\sum_{k=i}^{N} m_k g}$ is the shear coefficient at the $i$-th storey, $\alpha_1 = \frac{Q_{b,1}}{\sum_{k=1}^{N} m_k g}$ is the shear coefficient at the base, where $Q_{bi}$ is the shear force at the $i$-th story taken by the EDB system, $Q_{b1}$ is the shear force at storey 1 taken by the EDB system, and $\sum_{k=1}^{N} m_k g$ is the weight of the masses $m_k$ above the $i$-th story; then, $I_i = \frac{\sum_{k=i}^{N} m_k g}{\sum_{k=1}^{N} m_k g}$.

Akiyama provides an equation to estimate $\bar{\alpha}_i$, as:

$$\bar{\alpha}_i = \begin{cases} 
1 + 1.5927x - 11.851x^2 + 42.58x^3 - 59.48x^4 + 30.15x^5 & x > 0.2 \\
1 + 0.5x & x \leq 0.2 
\end{cases}$$

where $x = \frac{i-1}{N}$, $i$ is the storey number and $N$ is the total number of storeys of the frame.

3.2 Dissipation capacity

In retrofitting of gravity-load-designed structures, the dissipation capacity $E_p$ of the EDB systems is designed to balance the hysteretic energy demand $E_H$, so to maintain the existing structure elastic. The objective is to fulfil Eq. (5) through Eq. (6) at each storey.

Fig. 1 shows the schematic of a frame storey equipped with a bracing system. Bare frame and EDB system are two springs in parallel, so that the coupled system response is obtained by adding the bare frame and EDB responses, as shown in Fig. 2a.

Considering Fig. 2b, $E_p$ is defined as follows:
\[ E_p = n_{eq} Q_b (d_{fy} - d_{by}) \]  

(10)

where \( Q_b \) is the shear force taken by the EDB system, \( d_{fy} \) is the yield displacement of the frame, \( d_{bu} \) is the ultimate displacement of the EDB system. The parameter \( n_{eq} \) is defined in (Manfredi, Polese, and Cosenza 2003) as the equivalent number of plastic cycles at the maximum value of plastic excursion that the system must undergo to develop a plastic strain energy equal to \( E_p \). It is also used to account for near-fault and far-field situations. The expression for \( n_{eq} \) given in (Manfredi, Polese, and Cosenza 2003) is:

\[ n_{eq} = 1 + c_1 I_d \frac{T_{NH}}{T_1} (R - 1)c_2 \]  

(11)

where \( I_d = \int_0^t \frac{\ddot{u}_g(t)^2}{PGA PGV} dt \), with \( \ddot{u}_g(t) \) the ground motion, \( PGA \) the peak ground acceleration, and \( PGV \) the peak ground velocity, \( T_{NH} \) is the initial period of the medium period region in the Newmark-Hall spectrum (Newmark and Hall 1982), \( T_1 \) is the fundamental period of the structure, \( c_1 \) and \( c_2 \) are coefficients that differ for near-fault and far-field effects, and \( R \) is the strength reduction factor.

### 3.3 Required strength and stiffness of the EDB system

The objective stated in Eq. (6) requires that, at each \( i \)-th storey, the plastic strain energy \( E_{p,i} \) in Eq. (10) be equal to the corresponding hysteretic energy demand \( E_{H,i} \). From this energy balance, we obtain the shear force at each \( i \)-th storey in the EDB system:

\[ Q_{b,i} = \frac{E_{H,i}}{n_{eq}(d_{fy,i} - d_{by,i})} \]  

(12)

Notice that it is implicitly assumed that \( n_{eq} \) is the same at all storeys.

In order to compute \( Q_{b,i} \), it is necessary to know \( E_{H,i} \). That is, it is necessary to know the distribution of the hysteretic energy demand among the storeys. This can be ascertained either using equations proposed in the literature (Ye, Cheng, and Qu 2009) or from non-linear time history analyses.

An alternative approach could be to use the shear coefficient ratio \( \bar{\alpha}_i \) in Eq. (8), which allows to express the shear force at the \( i \)-th story as:

\[ Q_{b,i} = Q_{b,1} \bar{\alpha}_i \]  

(13)

where for \( \bar{\alpha}_i \) the expression in Eq. (9) can be adopted and for \( Q_{b,1} \) the following expression is used:

\[ Q_{b,1} = \frac{E_{H,1}}{n_{eq}(d_{fy,1} - d_{by,1})} \]  

(14)

In Eq. (14), \( E_{H,1} \) is defined according to the equation proposed in (Akbaş and Shen 2003):

\[ E_{H,i} = E_H \left\{ \begin{array}{ll} \frac{2(N + 1 - i)}{N(N + 1)}, & N < 5 \\
\frac{2(N - i)}{N(N - 1)}, & N \geq 5 \end{array} \right. \]  

(15)

where \( E_H \) is the total hysteretic energy demand, computed using energy spectra, e.g., those proposed in (Decanini and Mollaioli 2001), and \( N \) is the number of stories.
The shear force $Q_{b,i}$ at each $i$-th story is then found by replacing Eq. (15) into Eq. (14) and then, in turn, replacing the latter into Eq. (13).

The EDB system, at each $i$-th story, can now be designed as follows:

a) its shear capacity should be equal to the shear force $Q_{b,i}$ in Eq. (13),
b) its ultimate displacement should fulfill $d_{bu,i} = d_{fy,i}$,
c) its yield displacement $d_{by,i}$ is found by dividing the ultimate displacement $d_{bu,i}$ by the ductility capacity $\mu_{b,i}$ of the EDB system at that storey: $d_{by,i} = d_{bu,i}/\mu_{b,i}$. Notice that, usually, the ductility is kept constant at each $i$-th story ($\mu_{b,i} = \mu_b$), where $\mu_b$ results from an iterative procedure, as explained in the following section,
d) its stiffness is determined as:

$$k_{b,i} = \frac{Q_{b,i}}{d_{y,i}}$$

(16)

This design procedure ensures that the energy dissipated by the EDB system at each story, $E_{p,i}$, is larger than the corresponding hysteretic energy demand $E_{H,i}$.

3.4 Required stiffness, ductility, and strength of the single bracing device

Once the stiffness $k_{b,i}$ of the EDB system is obtained from Eq. (16), the bracings can be designed. Usually, they are made up from two components, as shown in Fig. 3: a) an elastic component, b) a dissipative component. These two components are assembled as a series system.

![Fig. 3: Assemblage of an EDB, a) EDB assembled in series; b) Stiffness coupling of two parts of the EDB; c) Coupled stiffness of the EDB](image)

The stiffness $k_{e,i}$ of the elastic part is kept higher than the stiffness $k_{d,i}$ of the dissipative component, that is $k_{e,i} = m k_{d,i}$. This is done in order, both, to avoid buckling failure of the elastic component, and to allow the dissipative component to dissipate energy by undergoing inelasticity. Such stiffness $k_{d,i}$ can be obtained writing the stiffness of the series system:

$$\frac{1}{k_{b,i}} = \frac{1}{k_{e,i}} + \frac{1}{k_{d,i}}$$

(17)

and solving for $k_{d,i}$:

$$k_{d,i} = \frac{k_{e,i} k_{b,i}}{k_{e,i} - k_{b,i}} = k_{b,i} \frac{1 + m}{m}$$

(18)

The ductility $\mu_{d,i}$ can be found as:

\newpage
\[
\mu_{d,i} = \mu_b \frac{k_{d,i} + k_{e,i}}{k_{e,i}} = \mu_b \frac{1 + m}{m}
\]

where \(\mu_b\) is the EDB system ductility, which results from the iterative procedure shown in Fig. 4. It is worth noticing that in practical applications it is usually assumed \(m \geq 3\).

![Flow chart of the proposed design procedure](image)

Each single brace at the \(i\)-th story has a shear capacity \(Q_{b,i,s}\) and a stiffness \(k_{b,i,s}\), whose horizontal components are obtained considering, both, its angle \(\theta\) with respect to the horizontal (see Fig. 1), and the number \(n_i\) of braces placed at the \(i\)-th story, so to find:
\[ Q_{b,ls} = \frac{Q_{b,i}}{n_i \cos \theta} \quad (20) \]
\[ k_{b,ls} = \frac{k_{b,i}}{n_i \cos^2 \theta} \quad (21) \]

where \( Q_{b,i} \) is in Eq. (13) and \( k_{b,i} \) is in Eq. (16).

The flow chart in Fig. 4 summarizes the proposed design procedure. It is worth noticing that the first step of the procedure (compute \( d_{fy,i} \) at each storey) requires carrying out a pushover analysis for each single storey, by fixing all the others. This provides a reasonable estimate of \( d_{fy,i} \). The so-obtained values of \( d_{fy,i} \) represent the target interstorey drifts at each storey, which should not be exceeded in the braced frame. Ideally, after applying the EDB system, all storeys should attain an interstorey drift as close as possible to \( d_{fy,i} \). This would allow an even energy dissipation distribution along the different storey, thus maximizing \( E_p \). In a sense, this implies that the top displacement of the braced frame should be as close as possible to:

\[ d_t = \sum_{i=1}^{n} d_{fy,i} \]

### 4 Comparison of the proposed design procedure with selected methods from the literature

Three reinforced concrete frames, denoted as Case 1 (3 storeys), Case 2 (5 storeys), and Case 3 (9 storeys, irregular in height), shown in Fig. 5, have been studied. Each frame has bay length of 5.5 m and story height of 3 m. Cross-sectional information is given in Fig. 5. The frames are modelled with linear elastic elements with concentrated end plastic hinges. The elastic stiffness of beams and columns is reduced by a factor of 0.4 and 0.5, respectively, to account for cracking, according to Eurocode-8 (Eurocode 8: Design of structures for earthquake resistance 2005). The columns are fixed at the base, and the beam-column joints are considered as infinitely rigid. The frames are subjected to non-linear static analysis for obtaining the yield interstorey drifts of the existing frame and then to non-linear dynamic analysis to determine the performance of the braced frame, where Rayleigh’s damping is used to account for the dissipation in the EDB system. Steel and concrete strength is 370 and 19.5 MPa, respectively. The force-displacement constitutive law of energy EDBs is defined using multi-linear link elements and the kinematic hysteresis model is employed for non-linear analysis.

![Fig. 5: Layout of 2-D case study RC frames](image-url)
4.1 Seismic action

The seismic action is defined through the acceleration response spectrum given in (Ministero delle infrastrutture e dei trasporti 2018) and the total hysteretic energy demand was estimated using the energy spectra proposed in (Decanini and Mollaioli 2001). The frames are analysed under a design earthquake of $PGA = 0.3g$.

4.2 Performance assessment of the bare frames

The bare frames performance is assessed through pushover analysis. The lateral load follows the first mode shape. From the deformed shapes in Fig. 6 and the interstorey drift profiles in Fig. 7, it is observed that soft-story mechanisms are formed in Case 1, at the base, and in Case 3, at mid height due to the irregularity; in Case 2 damage is distributed along the height, with a higher concentration at storey 1. The interstorey drift profiles highlight the need for an intervention aiming at obtaining a more uniform drift distribution, as presented in the following section.

Fig. 6: Deformed shapes of the bare frames; a) case 1 (3 storeys); b) case 2 (5 storeys); c) case 3 (9 storeys)

Fig. 7: Drift profiles of the bare frames.
4.3 Performance assessment of the braced frames

In this section, the EDB systems are designed, both, with the selected literature methodologies and with the proposed procedure, and their relative effectiveness is compared by performing non-linear static and non-linear dynamic analyses of the braced frames shown in Fig. 8. The braces positions are determined based on architectural constraints and their properties are obtained by applying both the selected literature methodologies and the proposed procedure.

Fig. 8: Configuration of bracings in the braced frames

![Fig. 8: Configuration of bracings in the braced frames](image)

Fig. 9: Stiffness profiles of the EDB systems as resulting from the design according to the proposed and the selected procedures

![Fig. 9: Stiffness profiles of the EDB systems as resulting from the design according to the proposed and the selected procedures](image)

In Fig. 9 the stiffness profiles of the designed EDB systems are shown. It can be observed that:

a) the methodology by Benavent-Climent (BC in the figures) results in an irregular stiffness distribution along the frame height. This irregularity causes damage concentration at the relatively weaker storeys. In all three cases, the first storey remains significantly less stiff, while the stories above are provided with much higher stiffness, causing the damage to occur at the first storey with the consequent soft story mechanism. Moreover, in all three cases, the overall stiffness provided to the frames is comparatively much higher than for the other design methodologies, which implies that the EDB system will be more costly and less efficient,
b) Ponzo-Di Cesare (Ponzo in the figures) distribute the EDB system strength proportional to the story strength of the existing structure. The underlying concept is not to alter the strength profile along the structure height. However, in most existing frames, the strength profile is not regular, so that, after the application of the bracing system, such irregularity will remain, thus resulting in damage concentration at the weaker storeys, as shown in Fig. 11,
c) the proposed design methodology follows the optimum strength distribution concept, which makes sure that all stories contribute evenly to the demand energy dissipation, and as a result prevents damage concentration at weaker storeys. In a sense, this approach aims at regularizing the frame response.

4.3.1 Pushover analysis of the braced frames

Pushover analysis is a useful tool to assess the response of structures well into the non-linear range, prescribed by most modern codes (ATC-40 1996; Eurocode 8: Design of structures for earthquake resistance 2005; FEMA 356: Prestandard and Commentary for the Seismic Rehabilitation of Buildings 2000; Ministero delle infrastrutture e dei trasporti 2018) for assessing the capacity of existing buildings.

![Capacity curves and performance points of the braced frames](image)

**Fig. 10**: Capacity curves and performance points of the braced frames; a) Case 1; b) Case 2; c) Case 3

![Interstorey drift profiles from pushover analysis of the braced frames](image)

**Fig. 11**: Interstorey drift profiles from pushover analysis of the braced frames
The outcomes of the three procedures (shortly, BC, Ponzo, and proposed) are compared in Fig. 10, which shows the pushover curves of the braced frames along with the relevant performance points, and in Fig. 11, which shows the interstorey drift profiles. By comparing the outcomes in both figures, the different strategies of the three methods become apparent: BC aims mainly at stiffening the frame, to reduce the damage in the existing structure, while Ponzo aims mainly at energy dissipation by attaining larger displacements. This results in higher drifts in Ponzo’s case and lower drifts in BC’s case. However, both these drift profiles are irregular, with heavier damage in the existing structure localized at the weakest storeys. The proposed method is a compromise between the previous two strategies, mainly aiming at obtaining a higher global strength and a more regular drift profile. As a consequence, the performance point lays closer to $d_t$ (as defined in Eq. (22)), which ensures an optimal even distribution of energy dissipation in the EDB system.

4.3.2 Non-linear dynamic analysis of the braced frames

Non-linear dynamic analysis is carried out using real accelerogram recorded in Kobe, Japan scaled to match the design response spectra in the given site (Fig. 12). Fig. 13 shows the inter-storey drift profiles of the braced frames designed by the three methods (BC, Ponzo and proposed), from which the following observations can be drawn: a) BC gives rise to irregular strength and stiffness along the height of the structures, thus causing damage concentration at a single story in all three cases; b) Ponzo results in non-uniform drift profiles in all three cases, with higher damage concentration at the weakest storeys (bottom for Case 1 and 2, and mid-height for Case 3), which can be attributed to the variable ductility of the EDB system; c) the proposed procedure yields essentially uniform drift profiles, thanks, both, to a demand-proportional strength distribution, and to an equal ductility capacity at all storeys, which leads to an optimal energy dissipation along the storeys.

Fig. 12: Scaled Kobe earthquake accelerogram

Fig. 14 shows the ratio of the interstorey drifts obtained with the non-linear dynamic analyses and the pushover analyses. This is an indicator of the consistency of the design procedures, showing whether the drifts resulting from the pushover-based design are actually experimented by the frames under dynamic actions. The vertical dashed line in Fig. 14, placed at the ratio of one, represents the optimal consistency of the design method. In all three cases, the proposed method shows ratios that are closer to the optimal ratio, while BC and Ponzo show disperse and more scattered ratios, thus confirming a higher discrepancy between the pushover-based target drifts used in design and those attained in the actual dynamic response.

Furthermore, Fig. 15 shows the comparison of the storey-wise overall dissipated energy $E_{p,i}$ for the three cases designed with the proposed and the selected literature design procedures. The EDB systems designed according to BC dissipate almost the entire energy in the first storey, while the upper stories...
remain elastic with a few exceptions. Overall, the method results in insufficient energy dissipation. The Ponzo procedure provides higher energy dissipation for Cases 1 and 2. However, this occurs at the cost of damaging the existing structure, due to the attainment of larger drifts, as shown in Fig. 13, thus violating the pre-fixed threshold. In Case 3, the method results in insufficient energy dissipation.

The proposed procedure ensures an efficient energy dissipation throughout all storeys, keeping the storey drifts within the predefined threshold given in Eq. (22), thus maximizing the efficiency of the EDB systems. Furthermore, the shape of the dissipated energy $E_{p,i}$ is linear, with the slight exception for Case 3, which indicates that the designed strength provided by EDBs enhance both the strength and regularity of the frames.

![Fig. 13: Interstorey drift profiles from non-linear dynamic analysis of the braced frames](image)

![Fig. 14: Interstorey drift ratio between non-linear dynamic analysis and pushover analysis](image)
Fig. 15: Story-wise hysteretic energy dissipation

5 Conclusions

The study has been focused, first, on reviewing two design methodologies available in the literature for the design of passive EDB (energy-dissipative bracing) systems and, subsequently, on introducing a new energy-based method. The efficiency of the three methods has been validated on three reinforced concrete frames, retrofitted using such systems, whose performance has been compared through pushover and non-linear dynamic analyses. Based on the results obtained, the following conclusions can be drawn:

Story-wise stiffness and strength distribution

The procedures selected from literature are not particularly efficient in regularizing the response of the frames through the insertion of EDB systems, since the assigned additional stiffness and strength cause vertical irregularity of the frames. On the other hand, the proposed procedure provides additional strength to the storeys as a function of the demand, thus resulting in an optimal strength distribution.

Interstorey drifts

The drift profile obtained from pushover and non-linear dynamic analysis showed that the selected procedures from the literature fail to some extent, both, in distributing damage, and in achieving a uniform drift profile. The proposed method results in more uniform drift profiles and prevents damage concentration at a single story, thanks to the optimal distribution along the frame height of the EDB properties.

Story-wise energy distribution

Non-linear dynamic analyses showed that the selected literature procedures result either in energy dissipation being mainly concentrated at a single story or in lower global dissipation, while the proposed procedure ensures that energy dissipation occurs at all stories, thus maximizing the effectiveness of the EDB systems.

6 Declarations
**Author contribution:** All authors contributed to the study conception and design. Material preparation, data collection and analysis were performed by Raihan Rahmat Rabi, Vincenzo Bianco and Giorgio Monti. The first draft of the manuscript was written by Raihan Rahmat Rabi and all authors commented on previous versions of the manuscript. All authors read and approved the final manuscript.

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7 **References**

Akbaş, Bülent, and Jay Shen. 2003. “Seismic Behavior of Steel Buildings with Combined Rigid and Semi-Rigid Frames.” *Turkish Journal of Engineering and Environmental Sciences* 27(4): 253–64.

Akiyama, H. 1985. *Earthquake-Resistant Limit-State Design for Buildings*. University of Tokyo press.

ATC-40. 1996. 1 Seismic Evaluation and Retrofit of Concrete Buildings. https://www.atcouncil.org/vmchk/Rehabilitation-of-Engineered-Buildings/Seismic-Evaluation-and-Retrofit-of-Concrete-Buildings/flypage.tpl.html.

Benavent-Climent, Amadeo. 2011. “An Energy-Based Method for Seismic Retrofit of Existing Frames Using Hysteretic Dampers.” *Soil Dynamics and Earthquake Engineering* 31(10): 1385–96. http://dx.doi.org/10.1016/j.soildyn.2011.05.015.

Bertero, Raul D., and Vitelmo V. Bertero. 2002. “Performance-Based Seismic Engineering: The Need for a Reliable Conceptual Comprehensive Approach.” *Earthquake Engineering and Structural Dynamics* 31(3): 627–52.

Cesare, Antonio Di, and Felice Carlo Ponzo. 2017. “Seismic Retrofit of Reinforced Concrete Frame Buildings with Hysteretic Bracing Systems: Design Procedure and Behaviour Factor.” https://doi.org/10.1155/2017/2639361 (October 25, 2019).

Choi, H, and J Kim. 2006. “Energy-Based Seismic Design of Buckling-Restrained Braced Frames Using Hysteretic Energy Spectrum.” *Engineering Structures* 28: 304–11. https://www.sciencedirect.com/science/article/pii/S0141029605003056.

Chou, Chung Che, and Chia Ming Uang. 2000. “Establishing Absorbed Energy Spectra - An Attenuation Approach.” *Earthquake Engineering and Structural Dynamics* 29(10): 1441–55.

Dasgupta, Prabuddha, Subhash C Goel, Gustavo Parra-montesinos, and K C Tsai. 2004. “Performance-Based Seismic Design and Behavior of a Composite Buckling Restrained Braced Frame.” In *13th World Conference on Earthquake Engineering*, Vancouver, Canada.

Decanini, Luis D., and Fabrizio Mollaiaoli. 2001. “An Energy-Based Methodology for the Assessment of Seismic Demand.” *Soil Dynamics and Earthquake Engineering* 21(2): 113–37. https://www.sciencedirect.com/science/article/pii/S0267726100001020 (October 25, 2019).

“Eurocode 8: Design of Structures for Earthquake Resistance.” 2005. *European Committe for Standardization*.

**FEMA 356: Prestandard and Commentary for the Seismic Rehabilitation of Buildings.** 2000.

Housner, George William. 1956. “Limit Design of Structures to Resist Earthquakes.” In *Proceedings of The 1st World Conference on Earthquake Engineering*, 5.1-5.13. https://ci.nii.ac.jp/naid/10003998472/.

Manfredi, Gaetano, Maria Polese, and Edoardo Cosenza. 2003. “Cumulative Demand of the Earthquake Ground Motions in the near Source.” *Earthquake Engineering and Structural Dynamics* 32(12): 1853–65.
Ministero delle infrastrutture e dei trasporti. 2018. Gazzetta Ufficiale Aggiornamento Delle «Norme Tecniche per Le Costruzioni». Rome, Italy.

Newmark, NM, and WJ Hall. 1982. *Earthquake Spectra and Design*. Earthquake Engineering Research Institute.

Riddell, Rafael, and Jaime E. Garcia. 2001. “Hysteretic Energy Spectrum and Damage Control.” *Earthquake Engineering and Structural Dynamics* 30(12): 1791–1816.

Shen, J., and B. Akbas. 1999. “Seismic Energy Demand in Steel Moment Frames.” *Journal of Earthquake Engineering* 3(4): 519–59.

Ye, Lieping, Guangyu Cheng, and Zhe Qu. 2009. “Study on Energy-Based Seismic Design Method and Application on Steel Braced Frame Structures.” In *Sixth International Conference on Urban Earthquake Engineering*, Tokyo, Japan.

Ye, Lieping, Guangyu Cheng, Zhe Qu, and Xinzheng Lu. 2012. “Study on Energy-Based Seismic Design Method and Application on Steel Braced Frame Structures.” In *Sixth International Conference on UrbanEarthquake Engineering*, Tokyo, Japan, 36–45.