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 Ponso-Di Cesare (Cesare and Ponso 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:

$$m\ddot{u} + c\dot{u} + f_s = -m\ddot{u}_g \quad (1)$$

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:

$$\int_0^{t_0} m\ddot{u}\dot{u}dt + \int_0^{t_0} c\dot{u}^2dt + \int_0^{t_0} f_s\dot{u}dt = - \int_0^{t_0} m\ddot{u}_g\dot{u}dt \quad (2)$$

where each term can be written as:

$$E_k + E_\xi + E_s = E_I \quad (3)$$

where E_k is kinetic energy, E_ξ 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:

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

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:

$$E_p \geq E_H \quad (5)$$

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

$$E_{p,i} \geq E_{H,i} \quad (6)$$

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

$$d_{bu,i} \leq d_{fy,i} \quad (7)$$

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:

$$\bar{\alpha}_i = \frac{\alpha_i}{\alpha_1} = \frac{Q_{b,i}}{Q_{b,1}} \Gamma_i \quad (8)$$

111 where $\alpha_i = Q_{bi}/\sum_{k=i}^N m_k g$ is the shear coefficient at the i -th storey, $\alpha_1 = Q_{b1}/\sum_{k=1}^N m_k g$ is the shear
 112 coefficient at the base, where Q_{bi} is the shear force at the i -th story taken by the EDB system, Q_{b1} is
 113 the shear force at storey 1 taken by the EDB system, and $\sum_{k=i}^N m_k g$ is the weight of the masses m_k
 114 above the i -th story; then, $\Gamma_i = \sum_{k=1}^N m_k g / \sum_{k=i}^N m_k g$.

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

$$\bar{\alpha}_i = \begin{cases} 1 + 1.5927x - 11.851x^2 + 42.58x^3 - 59.48x^4 + 30.15x^5 & x > 0.2 \\ 1 + 0.5x & x \leq 0.2 \end{cases} \quad (9)$$

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

117
118 Fig. 1: Schematic of a frame equipped with an EDB system: a) gravity-load supporting frame; b) EDB system; c) frame + EDB system

119 3.2 Dissipation capacity

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

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

126
127 Fig. 2: Coupled system composed of frame and EDBs: a) force-displacement curves; b) energy-dissipation by existing frame and EDBs
128

129 Considering Fig. 2b, E_p is defined as follows:

$$E_p = n_{eq} Q_b (d_{fy} - d_{by}) \quad (10)$$

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:

$$n_{eq} = 1 + c_1 I_d \sqrt{\frac{T_{NH}}{T_1}} (R - 1)^{c_2} \quad (11)$$

where $I_d = \int_0^t \frac{\ddot{u}_g^2(t)}{PGA \cdot PGV} dt$, 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:

$$Q_{b,i} = \frac{E_{H,i}}{n_{eq}(d_{fy,i} - d_{by,i})} \quad (12)$$

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:

$$Q_{b,i} = Q_{b,1} \frac{\bar{\alpha}_i}{\bar{\Gamma}_i} \quad (13)$$

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

$$Q_{b,1} = \frac{E_{H,1}}{n_{eq}(d_{fy,1} - d_{by,1})} \quad (14)$$

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

$$E_{H,i} = E_H \begin{cases} \frac{2(N+1-i)}{N(N+1)}, & N < 5 \\ \frac{2(N-i)}{N(N-1)}, & N \geq 5 \end{cases} \quad (15)$$

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 fulfil $d_{bu,i} = d_{fy,i}$,

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 μ_b results from an iterative procedure, as explained in the following section,

d) its stiffness is determined as:

$$k_{b,i} = \frac{Q_{b,i}}{d_{y,i}} \quad (16)$$

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

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:

$$\frac{1}{k_{b,i}} = \frac{1}{k_{e,i}} + \frac{1}{k_{d,i}} \quad (17)$$

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

$$k_{d,i} = \frac{k_{e,i} k_{b,i}}{k_{e,i} - k_{b,i}} = k_{b,i} \frac{1 + m}{m} \quad (18)$$

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

$$\mu_{d,i} = \mu_b \frac{k_{d,i} + k_{e,i}}{k_{e,i}} = \mu_b \frac{1 + m}{m} \quad (19)$$

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

Fig. 4: Flow chart of the proposed design procedure

181 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
 182 components are obtained considering, both, its angle θ with respect to the horizontal (see Fig. 1), and
 183 the number n_i of braces placed at the i -th story, so to find:
 184
 185
 186

$$Q_{b,i,s} = \frac{Q_{b,i}}{n_i} \frac{1}{\cos \theta} \quad (20)$$

$$k_{b,i,s} = \frac{k_{b,i}}{n_i} \frac{1}{\cos^2 \theta} \quad (21)$$

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: (22)

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

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

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

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

- 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.

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

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

Fig. 14: Interstorey drift ratio between non-linear dynamic analysis and pushover analysis

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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