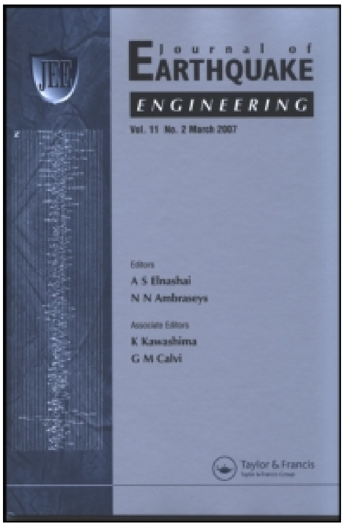

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Plastic Design of Seismic Resistant V-Braced Frames

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In this article, a new method for designing chevron concentrically braced steel frames is presented. The aim of the proposed method is the design of concentrically braced steel frames able to guarantee, under seismic horizontal forces, a collapse mechanism of global type. This result is of great importance in the seismic design of structures, because local failure modes give rise to a worsening of the energy dissipation capacity of structures and, therefore, to an higher probability of failure during severe earthquakes. With reference to the examined structural typology, the global mechanism is characterized by the yielding of tensile bracing diagonals and by the buckling of the compressed diagonals of all the stories.

The proposed method is rigorously based on "capacity design approach" which requires that dissipative zones have to be designed to withstand the internal actions due to the seismic design horizontal forces and the vertical loads acting in the seismic load combination; while non dissipative zones have to be designed considering the maximum internal actions that dissipative zones, yielded and strain-hardened, are able to transmit. The new design issue covered by the proposed design procedure is the need to account for the contribution of the compressed diagonals in deriving the design axial force of non dissipative members.

The seismic inelastic response of a sample structure is investigated by means of nonlinear dynamic analyses. The results carried out with reference to braced frames designed according to the proposed procedure are compared with those obtained with reference to the same structural schemes designed according to Eurocode 8.

Keywords Cyclic Behavior; Chevron-Braced Frames; Global Mechanism; Capacity Design; Design Methodology

1. Introduction

Concentrically braced frames (CBFs) are a popular lateral load resisting systems in high-risk seismic areas because of their economy, easy construction, and favorable stiffness. Because of the obstructions caused by cross-braces, chevron braces are often used to allow for door and windows openings. While the fulfillment of serviceability limit state is easy to obtain with reference to the considered structural typology, some uncertainty arises about the adequacy of such structures to resist to strong seismic actions by undergoing severe excursions in the nonlinear range. The energy dissipation capacity of CBFs is, in fact, almost completely related to nonlinear hysteretic behavior of diagonal braces under alternate tension and compression internal forces [Mazzolani *et al.*, 1994; Bruneau *et al.*, 1998; Mahin and Uriz, 2004]. This behavior is affected by a number of quite complex and not easily predictable aspects such as the performance of end connections, the in-plane, and out-of-plane overall buckling of compressed members and all the local damage phenomena (local buckling, low cycle fatigue, fracture propagation) related to

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the inelastic cyclic behavior under axial and bending forces. Therefore, in order to overcome the critical issues affecting the inelastic cyclic behavior of conventional braces under axial forces, the use of buckling restrained braces has been suggested [Sabelli *et al.*, 2003].

The energy dissipation capacity of a structure is strongly influenced by the kinematic mechanism developed at collapse. The need to prevent collapse mechanisms having limited dissipation capacity and to promote the development of a global failure mode is universally recognized. The problem of the failure mode control is faced by modern seismic codes by means of recommendations which are based on the simple hierarchy criterion. However, such design recommendations do not lead to structures failing in global mode and, for some structural typologies, they are not able to avoid the development of soft story mechanisms. Therefore, aiming to the design of structures able to assure the development of a collapse mechanism of global type under destructive seismic actions, more sophisticated design procedures have to be defined [Mazzolani and Piluso, 1997; Longo *et al.*, 2005a].

The global inelastic response of typical V-braced frames, designed according to the current European provisions [CEN, 2003a] is usually affected by "soft-story" problems, due to the column out-of-plane buckling for low values of intensity measure of seismic input.

In this structural typology two main behavioral aspects need to be considered. The first one is constituted by the post-buckling strength of the V-braced scheme, which is subjected to a quick degradation under load reversals. The second aspect is constituted by the out-of-plane column buckling, which causes a premature non dissipative collapse.

The first issue is hard to improve, because it is a consequence of geometrical configuration and conventional design criteria. For this reason, innovative bracing schemes, such as the suspended zipper frame, have been proposed [Leon and Yang, 2003; Yang *et al.*, 2008]. Conversely, regarding out-of-plane column buckling, it can be delayed by properly designing the structure according to capacity design principles. To this scope, in this work a new method for designing chevron concentrically braced frames is presented. It is aimed to assure the yielding of the brace elements of all the stories, i.e., a global mechanism, and is based on the "capacity design" principles. In addition, a new design issue covered by the proposed design procedure is the need to account for the contribution of the compressed diagonals in deriving the design axial forces of non dissipative members. Finally, the seismic performances of CBFs designed according to the provisions suggested by Eurocode 8 [CEN, 2003a] and according to the proposed approach, have been evaluated by means of nonlinear dynamic analyses. Regarding the comparison between the obtained seismic performances, which depends on the adopted design approach, the results herein presented are based on preliminary analyses corresponding to the classical deterministic approach.

2. Brace Behavior and Modeling

The behavior of CBFs depends, primarily, on the ability of bracing member to withstand large inelastic displacement reversals without significant loss of strength and stiffness. The member response under tension is mainly affected by the material behavior, while in compression the behavior is governed by the occurrence of buckling.

These aspects have been investigated by numerous researchers over the last 20 years [Bruneau *et al.*, 1998; Maison *et al.*, 1980; Ikeda and Mahin, 1984; Tang and Goel, 1989; Maison, 1992; Medhekar and Kennedy, 1998; Lee and Bruneau, 2002; Tremblay, 2002].

These investigations have included both experimental and analytical studies and have identified that the key parameter affecting the hysteretic behavior of bracing components is the slenderness ratio λ. Concentric braces are often described as either slender, intermediate, or stocky depending on the value of the normalized slenderness $\bar{\lambda}(\bar{\lambda} = \lambda/\lambda_y$ with $\lambda_y = \pi\sqrt{E/f_y}$ where f_y is the yield stress and E is the Young modulus).

In particular, slender braces $(\bar{\lambda} \geq \sqrt{2})$ [Bruneau *et al.*, 1998] are characterized by a critical eulerian stress less than $f_y/2$, so that buckling occurs essentially in elastic range. Conversely, stocky braces $(\bar{\lambda} \leq 0,65)$ [Bruneau *et al.*, 1998] are characterized by the occurrence of buckling in plastic range, so that the parameters governing the local buckling of the section play an important role. Finally, intermediate braces $(0.65 < \bar{\lambda} < \sqrt{2})$ are characterized by the interaction between the member global behavior and the local section behavior. The hysteresis loops for braces with different slenderness ratios vary significantly and, as a consequence, the modeling of bracing members constitutes a very important issue in the seismic design and performance evaluation of concentrically braced frames.

In a typical hysteretic loop, several characteristic zones can be identified (Fig. 1). The first branch O-A is associated with the compressive loading of the brace. The brace

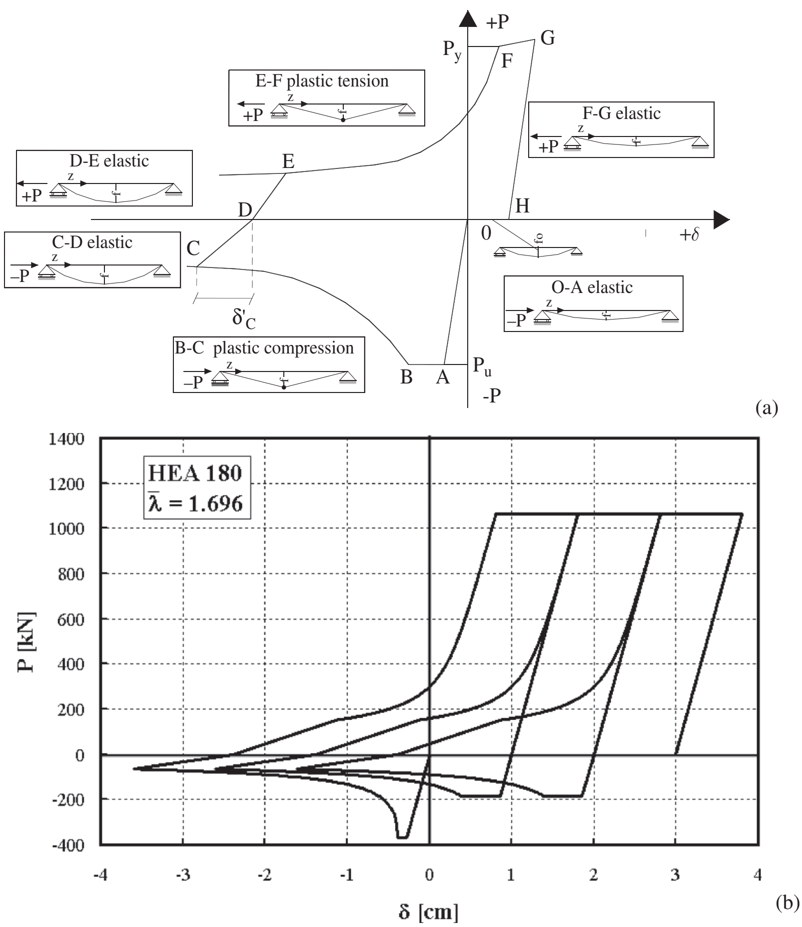

FIGURE 1 (a) Theoretical behavior for a pin-ended brace member; (b) degradation of cyclic brace behavior.

loaded in compression behaves elastically until it reaches the buckling condition (Point A). The buckling continues until the maximum moment in the member, given by the product between the axial load P and the lateral deflection, reaches its plastic value, i.e., until a plastic hinge is completely developed (Point B). After the formation of the plastic hinge, a kinematic mechanism, characterizing the post-buckling behavior, develops and the axial force decreases with the increase of the axial displacement δ which is kinematically related to the lateral deflection.

The branches C-D and D-E represent, respectively, the elastic unloading and reloading of the brace in tension. The slopes of these lines are less than the one of O-A branch, because the brace deflection is greater than the corresponding brace initial imperfection. As far as the tensile force increases (E-F branch), the brace elongates and gradually straightens. During this process, the bending moment in the midspan promotes a rotation of the initially formed plastic hinge in the opposite sense. At point F, the brace becomes straight and further lengthening to point G is caused by uniaxial yielding in the member. Due to the strain-hardening of material, the line F-G can develop a small positive slope. The following hysteresis loops have the same general characteristics, but the following changes occur:

- the starting point and the initial mid-span lateral deflection depend on the previous loading history;
- the buckling load significantly deteriorates with respect to the one corresponding to the first cycle. This aspect is governed by the Bauschinger effect deriving from the inelastic behavior of the material and by the residual deformation of the brace.

A great number of experimental data [Medhekar and Kennedy, 1998; Lee and Bruneau, 2002] shows that P_u quickly degrades after the first cycle. In particular, it can be assumed that the buckling load, in cycle following the first one, is equal to 50% of the initial one [Medhekar and Kennedy, 1998; AISC, 1993, 1994, 1997; CSA, 1994]; (Fig. 1b).

In this work, the Georgescu *et al.* [1992] model is used aiming to a quick yet accurate prediction of the hysteretic behavior of bracing members. This model allows a sufficiently accurate prediction of the cyclic response of brace elements. However, it cannot be directly applied for nonlinear dynamic analyses of braced steel frames, because most of available computer codes adopt, for sake of simplicity, a multilinear modeling of cyclic response. This is also the case of the PC-ANSR computer program, which has been adopted in the present work, where the Ikeda-Mahin model [Ikeda and Mahin, 1984] is implemented. Therefore, the nonlinear branches of the P-δ diagram, predicted by means of Georgescu model, have been linearized by introducing two new points, B' and E', which have been determined, as shown in Fig. 2, by imposing that the two models provide the same energy dissipation. As an example, Fig. 3 shows the comparison between Georgescu model [Georgescu *et al.*, 1992 and Ikeda-Mahin model [Ikeda and Mahin, 1984] for a section shape HE 200 B both with $\bar{\lambda} = 1.044$ and with $\bar{\lambda} = 0.617$.

3. Eurocode 8 Design Criteria

The structural scheme commonly adopted for evaluating the internal actions in beams, columns, and diagonals of V-braced frames subjected to seismic actions considers as active both the tensile diagonals and the compressed diagonals. In addition, a hinge connection is assumed between the bracing members.

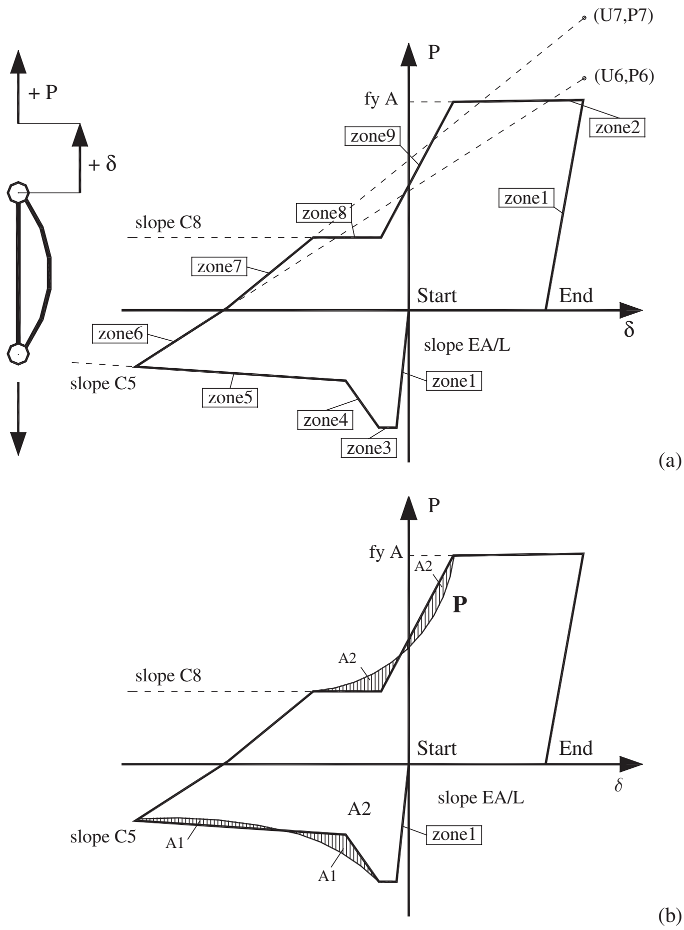

FIGURE 2 (a) Ikeda and Mahin model; (b) linearization of P-δ diagram for the analysis with PC-ANSR program.

The first issue to be faced for designing seismic-resistant structures is the selection of dissipative and non dissipative zones. Dissipative zones are those properly detailed to dissipate the earthquake input energy. Conversely, non dissipative zones are those whose premature collapse can prevent the complete exploitment of the plastic reserves that the structure is otherwise able to exhibit.

In the case of braced frames, the dissipative zones are constituted by the brace diagonal members. Conversely, beams, columns, and connections to foundations are non dissipative zones. Regarding the connections between the braces and the primary structure, the traditional design approach is based on the use of non dissipative connections; however, the EN version of Eurocode 8 [CEN, 2003a] has opened the door to the use of dissipative connections, but specific design criteria are not provided. The present work deals with the traditional approach of non dissipative connections.

According to Eurocode 8, the design resistance R_d of the connection of diagonals has to satisfy the following capacity design criterion:

$$R_d \geq 1.10 \cdot \gamma_{ov} \cdot N_{br,Rd} \tag{1}$$

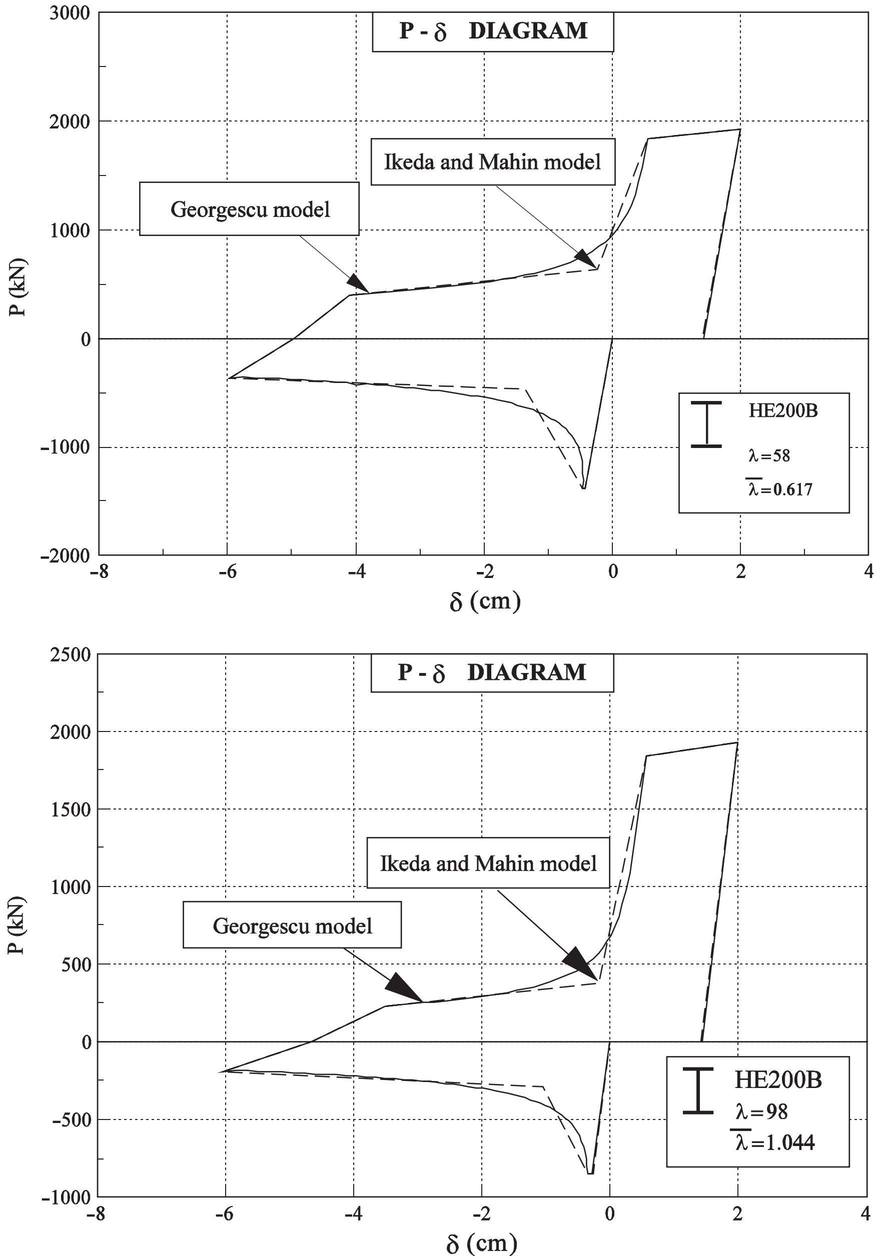

FIGURE 3 Comparison between Georgescu model and Ikeda-Mahin model.

where $N_{br,Rd}$ represents the design axial resistance of the bracing element, given by:

$$N_{br,Rd} = A_{br} \cdot f_y / \gamma_m \tag{2}$$

where A_{br} is the brace cross sectional area, f_y is the yield stress and γ_m is the partial safety factor. The value 1.1 in Eq. (1) is an amplification coefficient accounting for strain-hardening effects, while γ_{ov} is an overstrength factor accounting for the random variability of material properties and varying from 1.0–1.25.

In addition, Eurocode 8 [CEN, 2003a] requires that the normalized slenderness of braces has to be properly limited assuring that $\bar{\lambda} \leq 2.0$. The aim of this limitation is the reduction of the plastic out-of-plane deformation of the gusset plates, due to the brace buckling, which otherwise are prone to failure due to low-cycle fatigue. Even though the fulfilment of such limitation to the brace slenderness is desirable due to the need to avoid the fracture of gusset plates, it cannot be neglected that this slenderness limitation governs the overstrength of the diagonal bracing elements, especially at the top story, which governs the magnitude of beam and column axial forces to be considered into the design of such non dissipative members. As a consequence, the slenderness limitation provided by the codes should represent a compromise between two opposite needs: on one hand the preventions of gusset plate fracture by forbiding the use of excessively slender diagonals and, on the other hand, the need to reduce the overstrength governing the axial loads occurring in the beams and columns when first-brace yielding develops. The difficulty of such compromise is testified by the fact that the slenderness limitation has been changed many times during the Eurocode 8 development [CEN, 2000; 2003a].

Regarding the capacity design criterion for beams and columns, an overstrength coefficient Ω_i of bracing elements is preliminarily defined for each story:

$$\Omega_i = \frac{N_{br,Rdi}}{N_{br,Sdi}} \tag{3}$$

where $N_{br,Sdi}$ is the design value of the brace axial force and $N_{br,Rdi}$ is the corresponding design resistance. Eurocode 8 [CEN, 2003a] requires the fulfilment of the following relationship:

$$N_{Rd}(M_{Sd}) \geq N_{Sd,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Sd,E} \tag{4}$$

where N_{Rd} is the buckling resistance of the element (beam or column) according to Eurocode 3 [CEN, 2003b], $N_{Sd,G}$ is the axial force due to the non seismic loads included in the seismic load combination, $N_{Sd,E}$ is the axial force due to the seismic loads, M_{Sd} is the bending moment due to the seismic load combination, 1.1 is the amplification coefficient accounting for strain-hardening effects, γ_{ov} is the overstrength factor accounting for random variability of material properties, as described above, and Ω is the minimum value of the overstrength coefficients Ω_i as defined in Eq. (3): $\Omega = \min_{i=1}^{n} \Omega_i$ where n is the number of stories.

The value of γ_{ov} factor ranges from 1.0–1.25 with the aim of including all random effects of material properties. Therefore, considering the aim of such γ_{ov} factor, the accuracy of this design criterion suggested by Eurocode 8 should be investigated by means of Monte Carlo simulations. This work, conversely, is aimed to evaluate the nonlinear seismic response from a deterministic point of view; so that a γ_{ov} factor equal to 1.0 has been considered.

The term on right-hand side of Eq. (4) represents the axial force occurring in a non dissipative member (beam or column) when the first diagonal, corresponding to the story where $\Omega_i = \Omega$, reaches its ultimate resistance, i.e., when the first diagonal is completely yielded and strain-hardened in tension. Therefore, the design goal of Eq. (4) is to prevent column and beam buckling before the occurrence of yielding of at least one element, i.e., the one corresponding to the minimum value of Ω_i. However, it is useful to underline that the above design criterion does not assure the yielding of all the other diagonals, i.e. the participation of all the stories to the dissipation of the earthquake input energy [Longo, 2001; Montuori and Piluso, 2002].

Furthermore, it has to be underlined the design need to obtain the values of the Ω_i overstrength coefficient as uniform as possible with the aim to promote the brace yielding of more than one story, but this suggestion, also provided by Eurocode 8 is very difficult to be applied. For this reason, an innovative design approach, which is out of the scope of the present article, has been recently proposed [Longo *et al.*, 2005b] by introducing the concept of reduced section solution (RSS).

The beams have to be verified accounting for the vertical action resulting from the unbalanced brace axial forces, due to the fact that the compressed diagonal is buckled when the tensile one yields. In Eurocode 8, this force is approximately evaluated according to the following relationship:

$$V = (N_{br.Rd,t} \cdot \sin\alpha_1 - \gamma_{pb} \cdot N_{br.Rd,t} \cdot \sin\alpha_2) \tag{5}$$

where α_1 and α_2 are the angles between the diagonal axes and the beam axis (typically, $\alpha_1 = \alpha_2$), $N_{br.Rd,t}$ is design resistance of the diagonal in tension, and γ_{pb} is used for estimating the post buckling resistance of brace members. The value 0.3 is suggested.

It is important to underline that the design rule suggested for columns, i.e., Eq. 4), does not account for the column overloading due to the unbalanced brace axial force transmitted by beams to the columns.

4. The Proposed Design Methodology

According to the basic principles of capacity design, the aim of the proposed design methodology is the evaluation of the axial forces in non dissipative members (beams and columns) occurring when all the dissipative tensile diagonals are completely yielded and the compressed ones are buckled. Therefore, the evaluation of these forces is made by focusing the attention on the distribution of the internal actions occurring when a collapse mechanism of global type is completely developed.

The considered structural scheme is characterized by the column continuity, as shown in Fig. 4a, where the structure is considered in a deformed configuration, showing that the collapse mechanism is governed by only one parameter, i.e., the base rotation φ

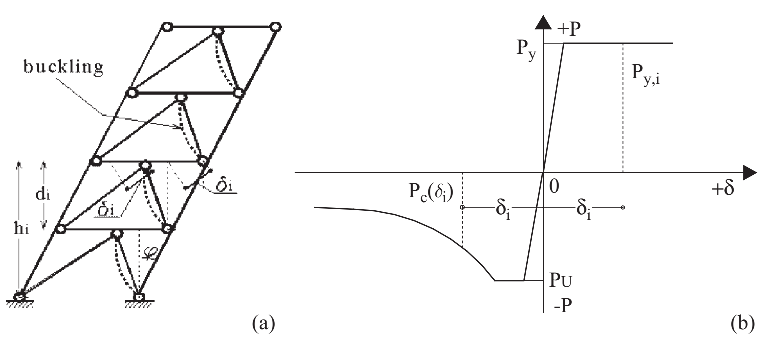

FIGURE 4 (a) V-braced frame in the deformed configuration corresponding to the global failure mode; (b) evaluation of compression and tension axial force.

of the structure where real hinges are located. The φ value corresponds to the yielding of all the diagonals in tension while the compressed ones are buckled. In addition, all the diagonals are pinned while the beams are considered continuous at the intersection with the diagonals and pinned at their ends.

Regarding the design procedure, it is assumed that the cross-sections of bracing members are known as they are designed to resist the internal axial forces due to the seismic load combination [CEN, 2003a]. Therefore, as a first step, the post-buckling response of bracing members can be properly predicted. The unknowns of the design problem are the beam and the column sections required to guarantee the yielding of all the tensile bracing members and the buckling of all the compressed diagonals, i.e., the participation of all the stories to the dissipation of the earthquake input energy.

The axial deformation corresponding to the yielding of the bracing member of the i-th story is given by:

$$\delta_{pi} = \frac{P_{yi} \cdot L_i}{E \cdot A_i} = \frac{f_y \cdot L_i}{E} \tag{6}$$

where P_{yi} is the yield load of the i-th diagonal, f_y is the yield stress, E is the elastic modulus, and L_i and A_i are the length and the cross-section area of the *i-th* diagonal, respectively.

For each story, it is possible to define the value of the interstory drift angle φ_i corresponding to the yielding of the i-th bracing member by means of the following yielding condition (Fig. 4a):

$$\delta_i = \varphi_i \cdot d_i \cdot \cos\alpha_i = \delta_{pi} \quad \text{for } i = 1, 2, ..., n \tag{7}$$

where d_i is the corresponding interstory height and α_i is the angle between the bracing member and the beam axis:

$$\varphi_i = \frac{\delta_{pi}}{d_i \cdot \cos\alpha_i} \quad \text{for } i = 1, 2, ..., n. \tag{8}$$

The value of the base rotation φ corresponding to the yielding of all the diagonals can be expressed as:

$$\varphi_m = \max_{i=1}^{n}(\varphi_i). \tag{9}$$

Therefore, for each story, the value of the axial deformation occurring in the diagonal members when the collapse mechanism is completely developed, can be easily computed as:

$$\delta_i = \varphi_m \cdot d_i \cdot \cos\alpha_i. \tag{10}$$

The above value provides both the elongation of the diagonal in tension and the shortening of the compressed diagonal subjected to buckling. Therefore, in the collapse condition, the axial force in the tensile diagonal of the *i-th* story is equal to P_{yi}, while the axial force in the corresponding buckled compressed diagonal can be derived according to Fig. 4b.

The net downward force resulting from the combinations of the forces acting in the diagonals of the *i-th* story is equal to:

$$V_i = P_{y,i} \sin \alpha_i - P_c(\delta_i) \cdot \sin \alpha_i. \tag{11}$$

The horizontal force transmitted by the braces to the beams is equal to:

$$H_i = P_{y,i} \cos \alpha_i + P_c(\delta_i) \cos \alpha_i. \tag{12}$$

The unbalanced action *(V_i)* applied to the beam, due to the post-buckling behavior of braces, is used both to design the beam and to evaluate the axial force in the columns, while the horizontal force H_i, applied in the midspan of the beam is used only in the beam design.

Therefore, according to capacity design principles, the design value of the axial force in the non dissipative members is computed by focusing the attention on the distribution of the internal actions occurring in the kinematic mechanism condition. In particular, with reference to the columns, the axial force to be considered in design is given by:

$$\begin{aligned} N_{c.Sd.E.i} &= \sum_{j=i+1}^{n} P_c(\delta_j) \cdot \sin \alpha_j + \frac{1}{2} \sum_{j=i}^{n} \left[P_{y,j} - P_c(\delta_j) \right] \cdot \sin \alpha_j + N_{c.Sd.G.i} \text{ for } i \leq n \\ N_{c.Sd.E.n} &= \frac{P_{y,n} \cdot \sin \alpha_n - P_c(\delta_n) \cdot \sin \alpha_n}{2} + N_{c.Sd.G.n} \text{ for } i = n \end{aligned} \tag{13}$$

where P_{yj} and $P_c(\delta_j)$ are the tension and compression forces in the *j-th* brace, respectively, while $N_{c.Sd.G.i}$ is the axial force in the column due to the gravity loads acting in the seismic load combination [CEN, 2003a].

It is important to underline that the beam axial forces are computed under the assumption that column shear forces are negligible.

5. Design Examples

The design methodology has been applied with reference to the four-story building depicted in Fig. 5. The seismic response of the structure designed as previously described

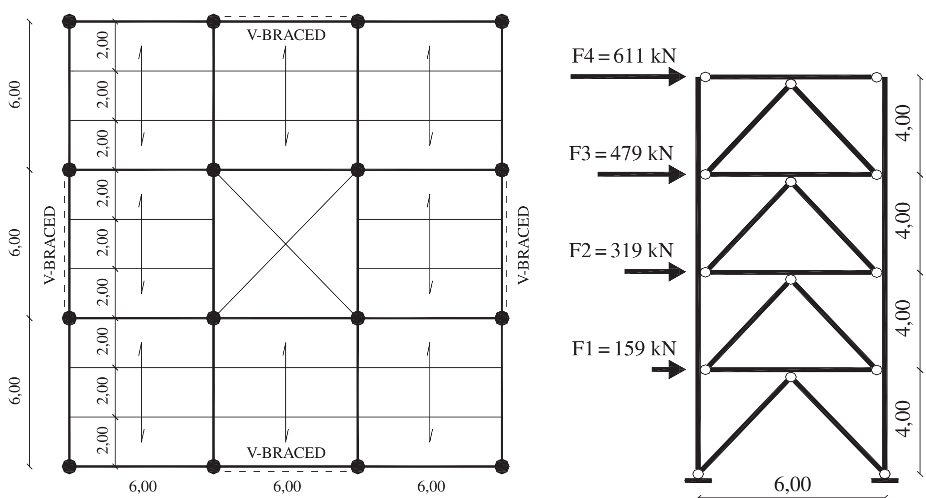

FIGURE 5 The analyzed structure.

has been compared with the one of the structural solution satisfying the provisions suggested by Eurocode 8. The structural layout is symmetrical in plan with two V-braced frames acting in each orthogonal direction. As a consequence, neglecting the accidental torsion due to the variability of location of live loads, the distribution of the seismic horizontal force among the vertical seismic resistant schemes is immediately derived. The story height at each level is equal to 4.0 m. The dead load is equal to 4 kN/m^2 and the live load is equal to 2 kN/m^2. S235 steel grade has been adopted. The total seismic action has been evaluated by means of the design spectrum given by Eurocode 8 for soil type A and for high seismicity zone with a peak ground acceleration a_g equal to 0.35 g. All the beam-to-column connections are pinned, therefore, all the seismic horizontal forces are withstood by the concentrically braced frames which are located along the perimeter of the structure.

The seismic base shear force is given by:

$$F_b = \frac{S_e(T_1)}{q} \cdot \frac{W}{g} = S_d(T_1) \cdot \frac{W}{g} \tag{14}$$

where $S_e(T_1)$ is the value of the elastic spectral acceleration corresponding to the fundamental period T_1 of the building ($T_1 = 0.05\ H^{3/4}$ being H the building height in meters, $T_1 = 0.40$ for the analyzsed case); a_g is the design peak ground acceleration; q is the behavior factor provided by Eurocode 8 and taken equal to 2.5; $S_d(T_1)$ is the value of the design inelastic spectral acceleration corresponding to the vibration period T_1 of the building; and W is the total seismic weight of the building.

The distribution of the seismic horizontal forces along the height of building, in the considered direction, has been evaluated by means of the relationship:

$$F_i = F_b \cdot \frac{z_i \cdot W_i}{\sum_{j=1}^{n} z_j \cdot W_j} = F_b \cdot \gamma_i \tag{15}$$

where F_i is the seismic horizontal force corresponding to the i-th story; F_b is the total seismic force, i.e., the design seismic base shear; and z_i is the height of the i-th structural level.

Finally, W_i is the seismic weight of the i-th story given by the following equation:

$$W_i = G_i + \varphi \cdot \psi \cdot Q_i \tag{16}$$

where G_i and Q_i are, respectively, the dead and the live load of the i-th story; ψ is a coefficient accounting for building usage and φ is a coefficient accounting for story location, assumed equal to one for all the stories. Table 1 summarizes the results for the four-story building depicted in Fig. 5 in terms of seismic weight and the total horizontal seismic forces determined by Eq. (15). The bracing diagonals have been dimensioned on the basis of the axial force due to the seismic horizontal forces and by considering the slenderness limitation required by Eurocode 8. Resulting from the above design requirements, the chosen profiles for the diagonal members are also reported in Table 1, where also the overstrength coefficients Ω_i previously defined and the values of the non dimensional slenderness $\overline{\lambda}_i$ are given. It is important to underline that the diagonal members are the same, both with reference to Eurocode 8 methodology and with reference to the proposed design methodology, because dissipative zones depends, according to the first principle of capacity design, on the code specified seismic forces

TABLE 1 Story seismic forces, diagonal, beam, and column member sections for the two design criteria

								VBF-E		VBF-P	
Story	z_i [m]	W_i [kN]	γ_i	F_i [kN]	Section diagonal	Ω_i	$\overline{\lambda_i}$	Beams	Columns	Beams	Columns
1	4	2267.40	0.1017	319.49	HEA 280	1.576	0.760	HEM550	HEB500	HEM500	HEB450
2	8	2267.40	0.2034	638.98	HEA 260	1.585	0.819	HEM500	HEB300	HEM500	HEB300
3	12	2267.40	0.3051	958.47	HEA 240	1.809	0.880	HEM450	HEB200	HEM500	HEB220
4	16	2171.20	0.3896	1223.74	HEA 200	2.241	1.060	HEM320	HEB100	HEM500	HEB160

only. In addition, it is useful to note that the maximum brace overstrength (Ω_i) corresponding to the selected brace sections does not satisfy the Eurocode 8 suggestion stating that such maximum overstrength should not differ from the minimum value by more than 25%. The reason for not complying with this design suggestion derives from the need to comply with the check against buckling, governing the Ω_i values at the stories different from the one corresponding to Ω_{min}. Conversely, the above difference could be reduced to values less than 25% by increasing the size of the diagonals of the stories different from the one corresponding to Ω_{max}, but this solution leads to a significant increase of the structural weight. In addition, further practical difficulties arise from the need to select the brace section from standard shapes. Table 1 provides also the beam and the column sections obtained from the application both of the proposed design procedure and of the provisions suggested by Eurocode 8. Furthermore, it is can be useful to point out that the frame designed according to the method proposed leads to first story columns (HEB 450) smaller than those (HEB 500) corresponding to the design carried out according to Eurocode 8. Even though this result could seem unexpected, it can be justified considering that the column axial force, in Eurocode 8, is governed by the brace overstrength Ω_{min} according to Eq. (4). This design rule leads to the underestimation of the axial force occurring, under destructive ground motions, in the columns belonging to the stories corresponding to $\Omega_i > \Omega_{min}$. Conversely, with reference to the story corresponding to Ω_{min} the differences with respect to the proposed method are due to the factor 1.10 given in Eq. (4) and to the need to choose the sections from standard shapes.

Finally, it is useful to underline that, as result of the member sections obtained by means of the analyzed design procedures, the actual period of vibration, obtained by means of dynamic analyses, is slightly different from the one (0.40) adopted for computing the design base shear, being equal to 0.505 for VBF-E and 0.495 for VBF-P.

The proposed design methodology provides an increase of the structural building weight and, as a consequence, an increase of the cost of the whole building when compared with the building designed according to Eurocode 8. With reference to the single-braced frame of the horizontal force resisting system, the increase in structural weight is equal to 1.38% while the increase with reference to the whole structural system, including leaning columns, is equal to 0.61% (Table 2). In addition, starting from the assumption that the cost of the structural elements is almost 30% of the cost of the whole building (included non structural elements), it is possible to evaluate the influence of the adopted structural design criteria on the whole building cost. In the examined case, the suggested design method leads to an increase of the whole building cost, when compared to Eurocode 8, equal to 0.20%.

TABLE 2 Comparison in terms of weight

	VBF-E	VBF-P
Weight of horizontal force resisting system (tons)	11.88	12.04
Weight of vertical loads resisting system (tons)	56.80	56.80
Total weight of the structure (tons)	104.30	104.96
Percentage variation	–	+0.61%

6. Results of Deterministic Analysis

A preliminary investigation of the seismic behavior corresponding to the different design solutions has been made by means of dynamic nonlinear analyses carried out by means of PC ANSR computer program. The bracing members are modeled using the nonlinear brace element with pinned ends, according to the previously defined model, while beams and columns are modeled using beam-column elements with the possibility of developing plastic hinges at their midspan and at their ends. A Newmark constant acceleration integration scheme has been used in this study. In addition, P-Δ effects are also considered. The vertical loads acting during the ground motion, according to the design seismic load combination, are equal to 100% of dead load and 30% of live load. Moreover, 5% of critical damping is adopted.

Each structure has been subjected to five historical ground motions (Table 3). All the records have been properly scaled to provide increasing values of the spectral acceleration $S_a(T_0)$ corresponding the first period of structures until collapse occurs. In this work, the collapse conditions has been defined considering three possible failure modes: the occurrence of out-of-plane buckling of columns, occurrence of fracture of braces, achievement of a drift limit value [FEMA 273, 1997], or the occurrence of dynamic instability. Conversely, the out-of-plane buckling of the beams is not allowed, because of the presence of the concrete deck.

As the computer code adopted for dynamic nonlinear analyses (PC-ANSR) accounts only for the in-plane behavior of the frame, the occurrence of buckling has been identified by means of the formulations given in Eurocode 3. To this aim, a specifically devoted post processor performing member stability checks at each time step of the time-history analysis has been developed [Longo *et al.*, 2005a].

Moreover, the state of damage of buildings subjected to severe ground motions is also strictly related to the maximum interstory drift ratio (MIDR) and to the ductility demand of bracing diagonals. The limit value of MIDR is not simple to establish. Different suggestions can be found in FEMA 273 [1997] and FEMA 350 [2000]. In the following, reference is made to the collapse prevention limit state by assuming a drift limit state value equal to 2%. Regarding the limit value of the brace ductility demand, it has been computed as a function of the normalized slenderness according to the following relationship [Tremblay, 2002]:

TABLE 3 Selected record and corresponding $S_{a,C/g}$ values at the collapse condition for analyzed structures

N	Record	Date	Component	a_{max}/g	Length [s]	VBF- E (Out-of-Plane buckling failure mode) $S_{a,C}$ /g	VBF- P (Dynamic instability failure mode) $S_{a,C}$ /g
1	IRPINIA, Italy	23/11/1980	N-S	0.133	72.61	0.75	1.20
2	EL CENTRO	19/05/1940	E-W	0.226	29.78	0.45	1.20
3	KOBE	16/01/1980	N-S	0.629	35.00	0.45	0.75
4	NORTHRIDGE	17/01/1994	N-W	0.842	15.55	0.50	0.55
5	TOKYO	09/05/1905	N-S	0.075	11.40	0.40	0.90
						0.51	**0.92**

$$\mu_u = a + b \cdot \bar{\lambda} \tag{17}$$

where $a = 2.4$ and $b = 8.3$.

On the basis of available experimental data, the above formulation was originally proposed with reference to the cyclic ductility (μ_c=$\delta_{c,max}$/δ_y where $\delta_{c,max}$ is the maximum cyclic displacement starting from load reversal and δ_y is the yield displacement). Notwithstanding, because of the gap of specific knowledge, in the present work it has been used both with reference to the ultimate value of the cyclic ductility and with reference to the ultimate value of the kinematic ductility. This is a conservative assumption (safe side solution) as widely explained in a another work by the same authors [Longo *et al.*, 2008a, b].

As a result, the evaluation of the seismic performances of the designed buildings has been carried out with reference to four parameters corresponding to four failure modes:

- out-of-plane buckling of columns, occurring when the parameter N_{max}/N_{Rd} (i.e., the ratio between the column axial force computed at any time step and the corresponding buckling resistance) for any column exceeds 1.0;
- fracture of diagonal braces, occurring when at any story, the brace ductility demand μ exceeds the brace ductility supply μ_{u} provided by Eq. (17) (i.e., the ratio μ /μ_u exceeds 1.0);
- excessive story damage, occurring when the maximum interstory drift ratio exceeds the limit value (0.02 rad);
- dynamic instability.

The spectral acceleration corresponding to the attainment of dynamic instability can be defined as the value for which a very little increase of $S_a(T_0)$ causes an exponential increase of all damage parameters leading to numerical instability and lack of convergence in dynamic analysis. Therefore, incremental dynamic analyses (IDA) have been performed by increasing the $S_a(T_0)$ value until the occurrence of dynamic instability. This phenomenon is due to the fact that, under load reversals, the previously buckled brace member could not return to its original alignment and, in addition, the brace member which was previously in tension could exceed its capacity in compression. As a consequence, both diagonal members could be in a buckled condition. Therefore, the dynamic instability phenomenon occurs when, at any story, both diagonals are in a buckled configuration so that a soft-story develops. In Fig. 6, with reference to the structure designed according to proposed methodology, the results of IDA analyses are depicted in terms of MIDR of the first story. This figure shows that the IDA curves have an horizontal asymptote when dynamic instability arises.

With reference to the above defined limit states, the different performances provided by the application of both Eurocode 8 design criteria and the proposed design methodology have been evaluated. The design provisions of Eurocode 8 lead to the worst behavior in terms of $S_a(T_0)$ value corresponding to the collapse condition (Table 3). In particular, with reference to the structure designed according to Eurocode 8 provisions (VBF-E), the $S_{a,\ C}$ value corresponding to the collapse condition is due to the out-of-plane buckling of columns. Conversely, in the case of the structure designed by means of the proposed methodology (VBF-P), the $S_{a,\ C}$ value corresponding to the collapse condition is related, for all the considered records, to the dynamic instability which occurs before the achievement of the limit values of the considered damage parameters.

With reference to VBF-E structure, column buckling arises for each ground motion for quite small values of S_a. In particular, collapse occurs before yielding of all the

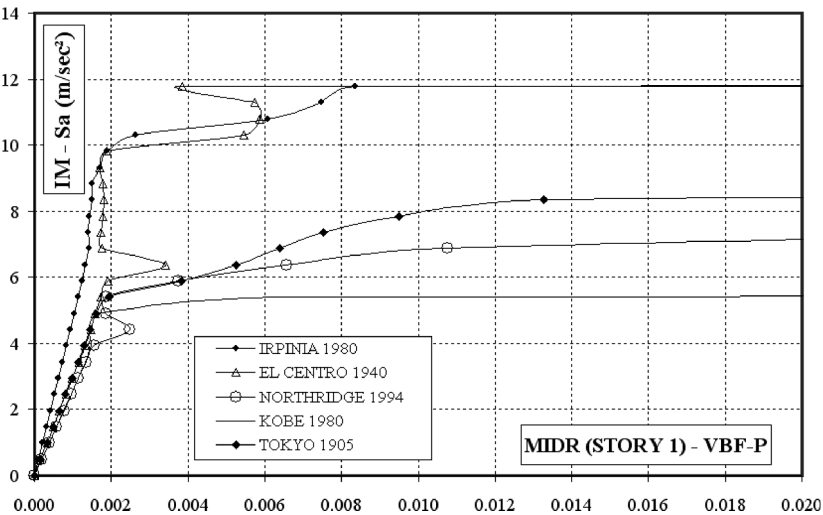

FIGURE 6 Interstory drift versus S_a curves resulting from IDA analyses.

diagonals. The premature failure of CBF designed by means of Eurocode 8 provisions is likely due to the underestimation of the axial force in the columns, because, in the current European provisions, the actions transferred by the beams due to the unbalance force arising from the brace post-buckling behavior are not considered.

A better seismic response is obtained for CBFs designed according to the proposed methodology. In Table 3, the $S_a(T_0)$ values corresponding the collapse condition are again reported for each ground motion. It is important to underline that, in terms of mean value, the $S_a(T_0)$ value leading to collapse for V-braced frames designed according to Eurocode 8 provisions is equal to 0.51 g, while it increases up to 0.92 g for structures designed according to the proposed design method. Furthermore, it is important to stress that the two design approaches lead to almost the same structural weight, but significantly different seismic performances.

Regarding the influence of record-to record variability, with reference to CBF designed according to the proposed method, it is useful to point out that unsatisfactory results are obtained with reference to Kobe and Northridge records. However, in these cases, the obtained results are essentially due to the spectrum shape of these records (Fig. 7) which is characterized by significant increase of the spectrum acceleration, with respect to the design spectrum, for periods of vibrations exceeding 0.50 (i.e., the actual initial period of vibration of the designed frames). Obviously, the affect of such spectral shape becomes more and more relevant with the structure period elongation resulting from damage.

With reference to Irpinia 1980 record, Fig. 8 shows the trend of $N_{max}/N_{b,Rd}$ damage parameter for each structure and for each story. From this figure it is possible to observe that for the structure designed according to the proposed procedure, the $N_{max}/N_{b,Rd}$ ratio is always less than one as far as $S_a(T_0)$ increases. It means that the primary goal of the proposed design procedure (VBF-P), i.e., the prevention of column buckling, is clearly satisfied. On the contrary, the application of Eurocode 8 design criteria (VBF-E) leads to the premature buckling of columns. The structure performance observed in Fig. 8 with reference to Irpinia record has been confirmed for all the records used for the dynamic analyses. Conversely, with reference to the other damage parameters the seismic behavior

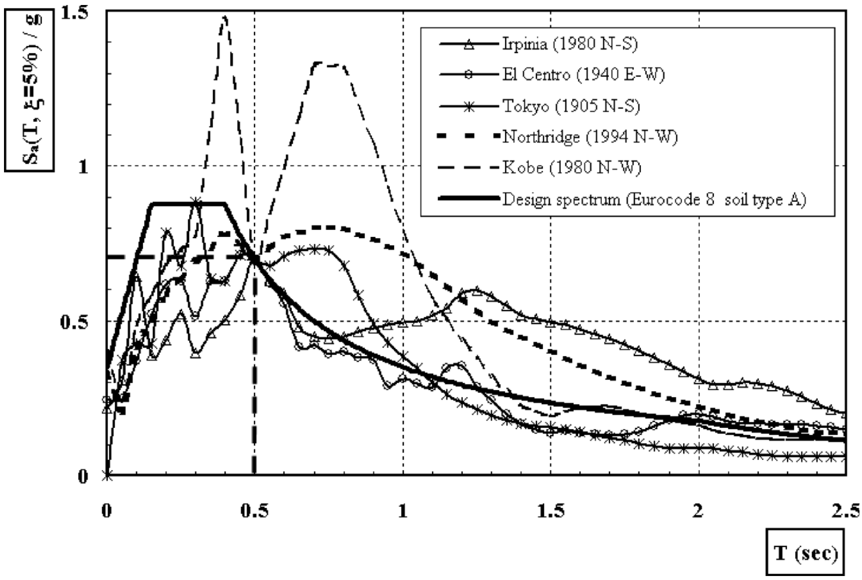

FIGURE 7 Comparison between selected records and design spectrum.

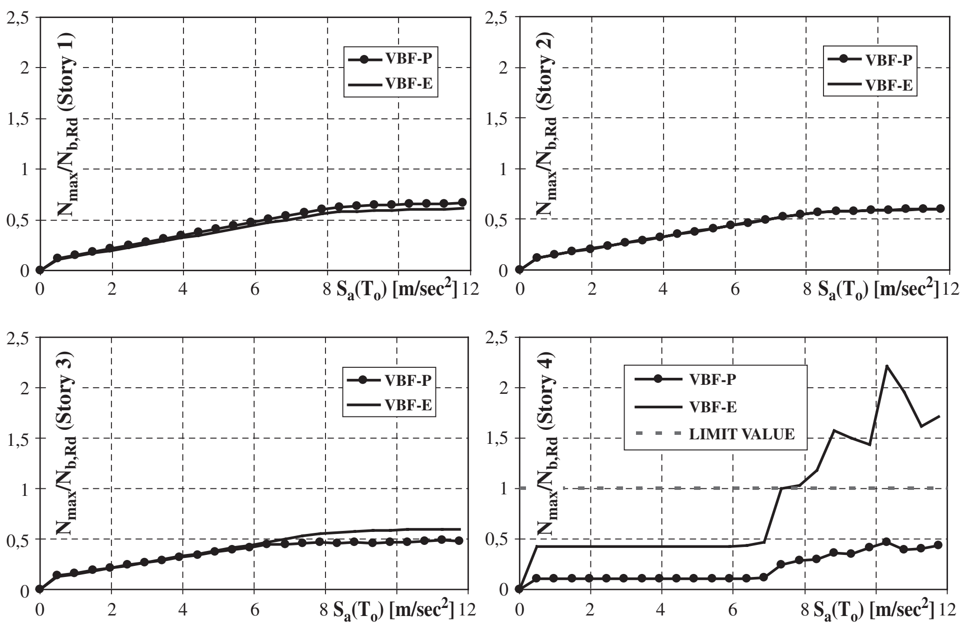

FIGURE 8 Trend of $N_{max}/N_{b,Rd}$ for the columns of each story and for Irpinia 1980 record.

of the designed structures (VBF-E and VBF-P) is almost the same. In particular, the values of MIDR and $\mu\ /\mu_u$ ratio do not reach the corresponding limit values. This behavior is testified in the Fig. 9 for MIDR and for $\mu\ /\mu_u$ damage parameters.

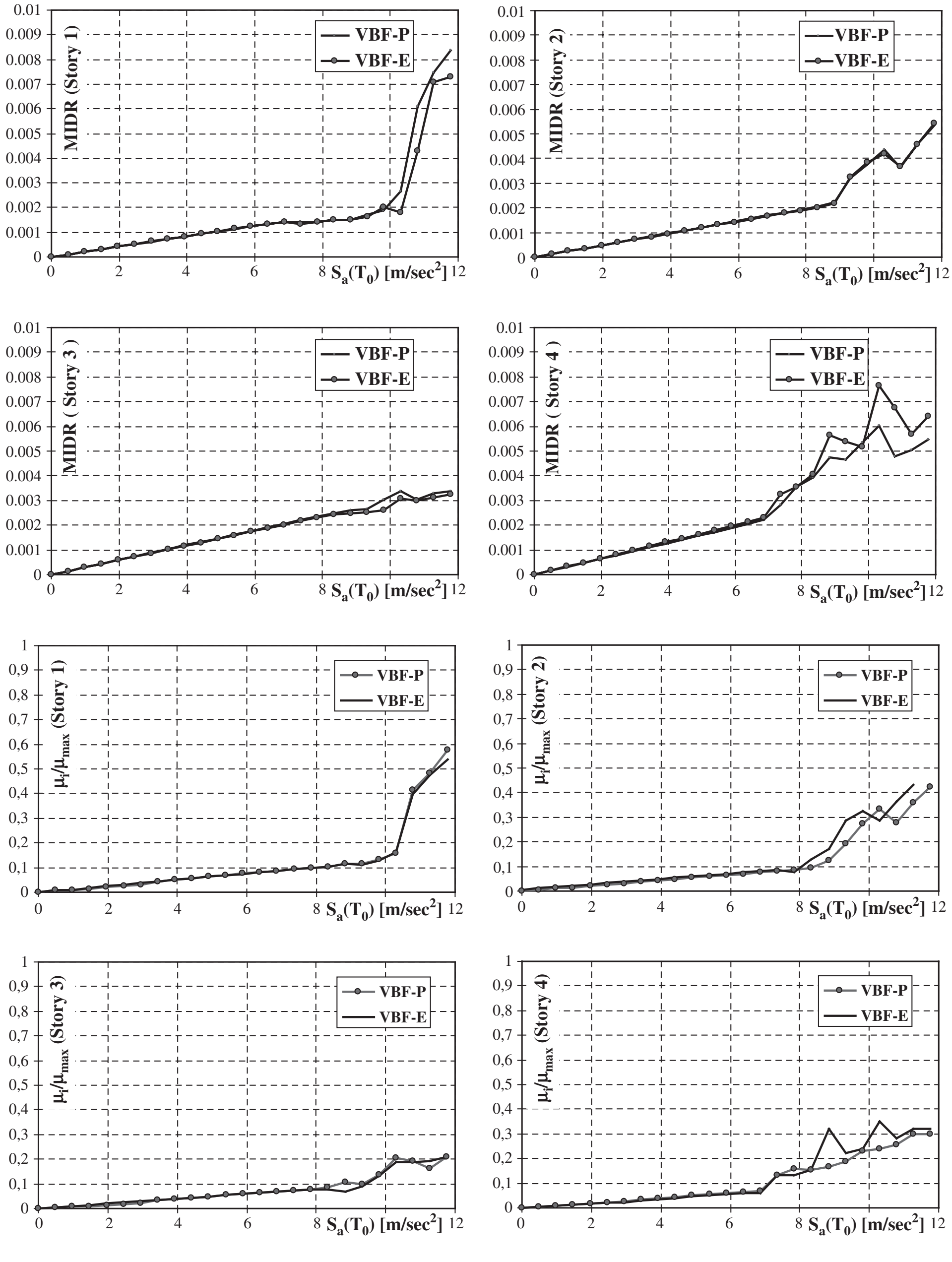

FIGURE 9 Trend of MIDR for each story and of μ_i/μ_{max} for the diagonals of each story for Irpinia 1980 record.

The failure of braced frames designed according the proposed method, for each ground motion considered, is due to dynamic instability. This phenomenon is due to the particular geometry of V-braced frames. In fact, it is well known that the post-buckling strength of V-braced schemes is subjected to a quick degradation under load reversals, because, the previously buckled member could not return in its original alignment and the member which was in tension could exceed its capacity in compression. As a consequence, both diagonal members could be in a buckled condition. Therefore, the dynamic instability phenomenon occurs when both diagonals of a story are in a buckled configuration, so that a soft-story develops.

Finally, it is important to underline again that structures designed according to Eurocode 8 and according to the proposed procedure have almost the same structural weight; in particular the difference is less than 1.5%. Therefore, the preliminary deterministic analyses herein presented show that, by means of the proposed design procedure, it is possible to obtain a significant improvement of the seismic behavior without a significant increase in the structural cost.

7. Conclusions and Future Developments

The results of the preliminary analyses presented in this article have pointed out that the application of the proposed design methodology provides a good seismic behavior when compared with that obtained by structures designed according to Eurocode 8 which leads to the premature buckling of columns.

The seismic response of CBFs designed by means of the proposed design procedure is characterized by the possibility to attain satisfactory values of the $S_a(T_0)$ value leading to collapse. The value obtained for the analyzed building is equal to 0.51 g for CBFs designed according to Eurocode 8 (VBF-E). Conversely, this limit increases up to 0.92 g for the examined structures designed by means of the proposed methodology (VBF-P). In addition, it is important to stress that the two design approaches lead almost to the same structural weight, but significantly different seismic performances.

The unsatisfactory seismic behavior of Eurocode 8 design methodology is primarily due to the underestimation of column axial load. In fact, the application of Eq. (4) does not assure the prevention of out-of-plane buckling of columns before the yielding of all the bracing members. In addition, Eurocode 8 provisions neglect the contribution to the column axial force due to the shear force transmitted by the beams arising from the unbalanced vertical actions due to the brace post-buckling behavior. This aspect is very important as demonstrated by the results of dynamic analyses.

Even though the preliminary performance assessment of the designed buildings is based on IDA analyses limited to only five records, the obtained results are encouraging about the performance improvements which can be attained by means of the proposed design procedure. However, it has to be recognized that seismic response of structures is highly affected by the frequency content of the ground motion, so that record-to-record variability has to be considered. This observation can be considered the base for the future development of the work which will require the application of a probabilistic approach aiming to evaluate the seismic reliability of such design criteria in terms of mean annual frequency of exceeding specified limit states.

Even though the influence of higher modes of vibration becomes more and mode relevant by increasing the members stories, it is useful to underline that results similar to those presented in this article have been obtained also in the case of taller bracing systems [Longo, 2005], not reported here.

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