Journal of Constructional Steel Research 66 (2010) 611–621

Contents lists available at ScienceDirect

Journal of Constructional Steel Research

journal homepage: www.elsevier.com/locate/jcsr

Review

Plastic design of CB-frames with reduced section solution for bracing members

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ARTICLE INFO

Article history:
Received 27 June 2008
Accepted 11 January 2010

Keywords:
Concentrically braced frames
Design methodologies
Diagonal members
Reduced section solution
Dynamic analyses

ABSTRACT

The seismic behaviour of concentrically braced frames (CBFs) designed according to the current European provisions is unsatisfactory due to the premature out-of-plane buckling of columns. For this reason, a new design methodology, based on a rigorous application of "capacity design" criteria has been recently proposed. In addition, aiming at a reduction of the plastic out of plane deformations of gusset plates due to brace buckling and at the prevention of sudden impact load affecting connections at the end of the straightening phase, Eurocode 8 requires the limitation of the brace slenderness. This limitation leads to the oversizing of diagonals and, consequently, of beams and columns. Therefore, to avoid this problem a new design strategy for bracing members is suggested: the Reduced Section Solution (RSS). It allows the calibration of the diagonal yielding resistance, leaving the brace slenderness practically unchanged.

The results of dynamic inelastic analyses carried out with reference to braced frames designed according to the proposed procedure, both with and without RSS, are compared with those obtained with reference to the same structural schemes designed according to Eurocode 8. The obtained results show that the proposed design approaches are able to assure a significant improvement of the seismic performance.

Contents

1. Introduction

During last decades, the analysis of the dynamic behaviour of steel structures has assumed a more and more important role in engineering research. The increasing interest is due to the numerous advantages offered by these structures in terms of low weight, resistance and ductility. Structural ductility, in particular, is an important property in seismic design, because it allows one to reduce the design horizontal forces, as recognised by the actual code provisions [1,2] and as pointed out by modern design philosophy of Performance Based Seismic Design (PBSD) [3–5]. Nevertheless, in order to assure a considerable structural ductility, the use of a ductile material, such as steel, is not sufficient but the development of a collapse mechanism of global type is also necessary.

Concentrically braced frames (CBFs) are very frequently used as seismic-resistant schemes. They are characterised by considerable lateral stiffness, which easily allows one to fulfil the serviceability requirements. On the other hand, some problems arise with reference to their capacity of assuring a reliable dissipative response under extreme earthquake conditions. These drawbacks are mainly due to the inelastic behaviour of bracing members, which is affected by local and/or global buckling phenomena. As a consequence, CBFs cannot rely on stable dissipation mechanisms and this adversely affects the overall dissipation capacity of the structure. In addition, CBFs designed according to current European provisions are affected by "soft-storey" problems. In fact, aiming at the safeguard of brace-to-column connections, Eurocode 8 [1] provides a limitation to the normalised slenderness of bracing members which, unfortunately, leads to their oversizing, especially at the upper storeys. As a result, a concentration of seismic input energy is obtained at the storey where the diagonal with the minimum overstrength is located, leading to the development of a soft-storey mechanism.

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doi:10.1016/j.jcsr.2010.01.001

For this reason, a new design methodology based on a rigorous application of the "capacity-design" philosophy is herein proposed [6,7]. It consists of designing dissipative members (i.e. diagonals) on the basis of internal actions arising under the seismic load combination, whereas non-dissipative elements (i.e. beams and columns) are designed by computing the axial forces that bracing members, yielded and strain hardened, are able to transmit considering the development of a global mechanism. The proposed methodology leads to a significant improvement of the seismic behaviour, because it completely avoids local failure mechanisms [6,7]. Nevertheless, because of slenderness limitations, Eurocode 8 provisions do not allow one to obtain a uniform involvement of the bracing members in energy dissipation. Therefore, aimed at the yielding of all storey's bracing members, a new strategy for brace design, namely the "Reduced Section Solution" (RSS), has been proposed [8]. The idea is similar to the use of "dog-bones" in moment resisting frames, because it is based on the reduction of the cross section area at the ends of the bracing members aiming to calibrate the axial resistance to a value equal to the internal action occurring under seismic load combination. As a consequence, the oversizing of bracing members is avoided, leaving their slenderness substantially unchanged.

In this paper, a comparison between seismic performances of CBFs designed by means of Eurocode 8 provisions and by means of the proposed methodology both with and without RSS is presented. Conversely, a comparison with other innovative bracing devices, such as buckling restrained braces, is outside the scope of the present paper and it will be the subject of future works. In this paper, a preliminary deterministic performance evaluation is carried out by means of non linear dynamic analyses for an increasing ground motion intensity (IDA—incremental dynamic analyse) using PC-ANSR programme [9]. The results coming from IDA analyses are presented and discussed.

2. Eurocode 8 design criteria

Eurocode 8 [1] rules require the design of dissipative structures such that yielding and local buckling or other phenomena, due to hysteretic behaviour, do not affect the overall stability of the structure. According to the "capacity design" philosophy, plastic deformations have to be located in dissipative zones to be designed with sufficient ductility, whereas the yielding of non-dissipative zones has to be prevented. Therefore, non-dissipative zones are designed by guaranteeing a sufficient overstrength to remain in the elastic range after the development of hysteretic cycles in the dissipative zones.

With reference to concentrically braced frames (CBFs), dissipative zones are identified by brace diagonal members which dissipate the seismic input energy by means of their axial cyclic behaviour which is characterised by yielding in tension and buckling in compression. Therefore, according to the described general rules, these structures are designed so that the yielding of diagonals in tension takes place before the failure of connections and yielding or buckling of columns or beams.

Dissipative elements, i.e. bracing members, have to be designed considering the internal actions occurring under the seismic load combination:

$$N_{pl.Rd} \geq N_{br.Sd} \tag{1}$$

where $N_{br.Sd}$ is usually determined neglecting the resistance of compressed diagonals, because of their high slenderness, while $N_{pl.Rd}$ is the design axial resistance of bracing elements in tension, given by:

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

where A_{br} is the brace cross section area, f_y is the yield stress and γ_m is the partial safety factor.

In addition, Eurocode 8 provides the following limitation to the brace slenderness:

$$1.3 < \overline{\lambda} \leq 2.0 \tag{3}$$

where the non-dimensional slenderness $\overline{\lambda}$ is the ratio between the diagonal slenderness λ and the elastic limit slenderness $\lambda_y = \pi\sqrt{E/f_y}$. The lower bound assures that, according to code provisions, diagonal braces can be considered to be active in tension only, whereas the upper one is imposed to reduce the plastic out of plane deformations of the gusset plates due to brace buckling and to prevent a sudden impact load on the connections at the end of the straightening phase following the post-buckling behaviour.

In addition, to guarantee the protection of non dissipative parts, and to allow the development yielding in dissipative zones non dissipative zones have to be designed by imposing a sufficient overstrength. Therefore the resistance of connections, R_d, should satisfy the following limitation:

$$R_d \geq 1.10 \cdot \gamma_{ov} \cdot N_{pl.Rd} \tag{4}$$

where the factor 1.10 accounts for strain-hardening effects; γ_{ov} is the overstrength factor, accounting for the random variability of the material's properties; $N_{pl.Rd}$ is the design axial resistance of the connected bracing member. The value of γ_{ov} factor ranges from 1.0 to 1.25. Regarding γ_{ov} factor, it has to be pointed out that this paper aims to evaluate the seismic response of structures from a deterministic point of view, so that a value equal to 1.0 has been assumed.

The design of beams and columns according to Eurocode 8 requires the fulfilment of the following relationships:

$$N_{b.Rd}(M_{Sd}) \geq N_{b.Sd.G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{b.Sd.E} \tag{5}$$

$$N_{c.Rd} \geq N_{c.Sd.G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{c.Sd.E} \tag{6}$$

where $N_{b.Rd}(M_{Sd})$ and $N_{c.Rd}$ are the design buckling resistance respectively of the beams and columns, computed according to Eurocode 3 [10], taking into account the interaction with the bending moment M_{Sd} occurring in the seismic load combination; $N_{b.Sd.G}$ and $N_{c.Sd.G}$ are the axial forces in the beams and columns due to non seismic actions included in the seismic load combination; $N_{b.Sd.E}$ and $N_{c.Sd.E}$ are the axial forces in the beams and columns due to seismic actions; Ω is an overstrength coefficient defined as:

$$\Omega = \min \Omega_i \quad \text{with } \Omega_i = \frac{N_{pl.Rd.i}}{N_{br.Sd.i}} \tag{7}$$

where $N_{br.Sd.i}$ is the design value of the brace axial action and $N_{pl.Rd.i}$ is the corresponding design resistance. It can be noted that Ω_i expresses, for each bracing member, a measure of its overstrength with respect to the design axial force.

Aiming to promote the yielding of braces of all the storeys, Eurocode 8 suggests using a distribution of Ω_i coefficients, along the height of the structure, as uniform as possible, but this is almost impossible to obtain due to the need of satisfying the upper limit of the slenderness limitation given by Eq. (3). In fact, in the upper stories the design value $N_{br.Sd.i}$ (which is directly related to the storey shear value) of the diagonal axial force is rather small; conversely, due to the above-mentioned slenderness limitation, the corresponding design resistance $N_{pl.Rd.i}$ results in being very high. Therefore, the resulting values of Ω_i coefficients at the upper storeys are always greater than those corresponding to the lower stories.

In other words, on one hand, the slenderness limitation safeguards the brace-to-column connections, but, on the other hand, it leads to the oversizing of bracing members, especially at the upper stories. As a consequence, under seismic action, plastic deformations are concentrated in the storey where the minimum value of the overstrength factor occurs.

In fact, the right terms of Eqs. (5) and (6) represent the axial forces occurring in the beams and columns, respectively, when the first diagonal, i.e. the one characterised by the minimum overstrength factor Ω_i, is completely yielded and strain-hardened. Therefore, the described design criteria do not assure the yielding of all the diagonals and the involvement of all the storeys in the seismic energy dissipation, because a "soft-storey" mechanism occurs.

In order to obtain a collapse mechanism of global type, by assuring a distribution of the overstrength factors Ω_i as uniform as possible, an innovative design approach is herein proposed by introducing the concept of the Reduced Section Solution (RSS) [8], in the same fashion of "dog-bones", or the reduced beam section solution, suggested with reference to moment-resisting frames [11–13].

3. Reduced Section Solution (RSS)

Code provisions regarding normalised slenderness limitations lead to the oversizing of bracing members, especially at the upper storeys, preventing the development of a collapse mechanism of a global type. Therefore, in order to safeguard the brace connections still satisfying the slenderness limitation, but without oversizing bracing members, the Reduced Section Solution (RSS) [8] can be adopted. The idea is based on a reduction of the brace sections at the member ends, in order to calibrate their axial resistance to a value equal to the internal action occurring under the design seismic load combination, so that there is no overstrength and the brace slenderness remains substantially unchanged.

In the case of members with a variable section, as depicted in Fig. 1, the following relationship provides the buckling load [14]:

$$N_{cr,r} = \frac{\pi^2 EI}{L^2} \frac{1}{1 + \left(\frac{I}{I_r} - 1\right)\left\{2\frac{L_r}{L} - \frac{1}{\pi} \cdot \sin\left[\frac{\pi(L-2L_r)}{L}\right]\right\}} \quad (8)$$

where L is the diagonal total length, L_r is the length of the reduced section zone, I and I_r are the inertia moments of the whole and of the reduced section, respectively. By means of this equation, two relationships regarding the influence of the reduced section zone on the brace slenderness and on the buckling load, respectively, can be derived [8]:

$$\frac{\lambda_{eq}}{\lambda} = \sqrt{1 + \left(\frac{I}{I_r} - 1\right)\left\{2\frac{L_r}{L} - \frac{1}{\pi} \cdot \sin\left[\frac{\pi(L-2L_r)}{L}\right]\right\}} \quad (9)$$

$$\frac{N_{cr,r}}{N_{cr}} = \frac{1}{1 + \left(\frac{I}{I_r} - 1\right)\left\{2\frac{L_r}{L} - \frac{1}{\pi} \cdot \sin\left[\frac{\pi(L-2L_r)}{L}\right]\right\}} \quad (10)$$

where λ_{eq} and $N_{cr,r}$ are the overall slenderness and the buckling load, respectively, of the brace member with a RSS; λ and N_{cr} are the slenderness and the buckling load, respectively, of the same brace member without a RSS. The graphical representations of Eqs. (9) and (10) are depicted in Fig. 2; these curves show that for L_r/L less than 0.3 a reduction of the buckling resistance due to a RSS is less than 10% and the corresponding amplification of the brace slenderness is less than 5%.

Regarding design criteria for braces with a RSS, an important issue to be solved consists of establishing the length of the reduced section zones. Under this point of view, a lower bound and an upper bound can be identified [8].

Concerning the lower bound, taking into account that bracing members have to yield in tension, it can be suggested that the minimum length of the reduced section zone ($L_{r,\min}$) has to satisfy the limitation commonly adopted for coupon tensile tests [15]:

$$L_{r,\min} = 5.65 \cdot \sqrt{A_r} \quad (11)$$

where A_r is the reduced section zone area.

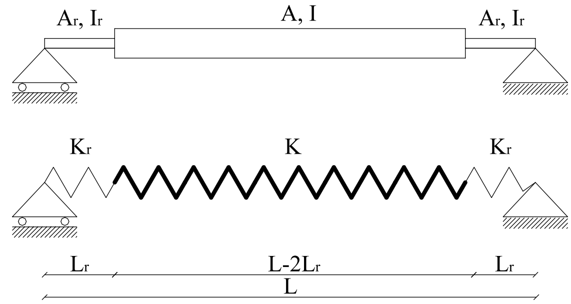

Fig. 1. Member with variable section.

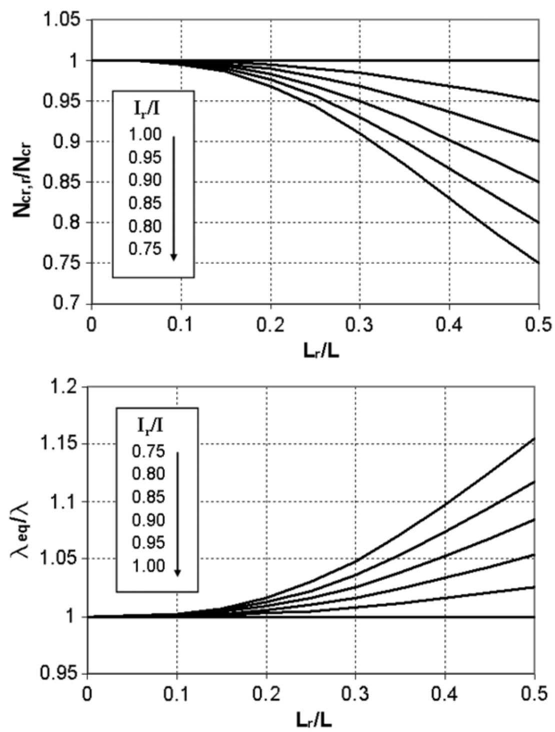

Fig. 2. Influence of reduced section on buckling load and brace slenderness.

Regarding the upper bound, i.e. the maximum length of the reduced section zone, it can be obtained by imposing that, during the post-buckling behaviour, a yielding of the midspan section occurs while the yielding of the reduced section zones is prevented. To this scope, a sinusoidal curve for the deflected shape of the buckled bracing member is assumed:

$$v(x) = f_0 \cdot \sin\frac{\pi x}{L} \quad (12)$$

where f_0 is the brace imperfection, L is the brace length and x is the generic abscissa of the bracing member. Accounting for the deflected shape, the second order bending moment is given by:

$$M(x) = N \cdot f_0 \cdot \sin\frac{\pi x}{L} \quad (13)$$

where N is the axial load. Therefore, the yielding condition of the reduced section zone can be written as:

$$N \cdot f_0 \cdot \sin\frac{\pi L_{r.\max}}{L} = M_{p.r}(N) \quad (14)$$

whereas the yielding condition of the midspan section is given by:

$$N \cdot f_0 = M_p(N) \quad (15)$$

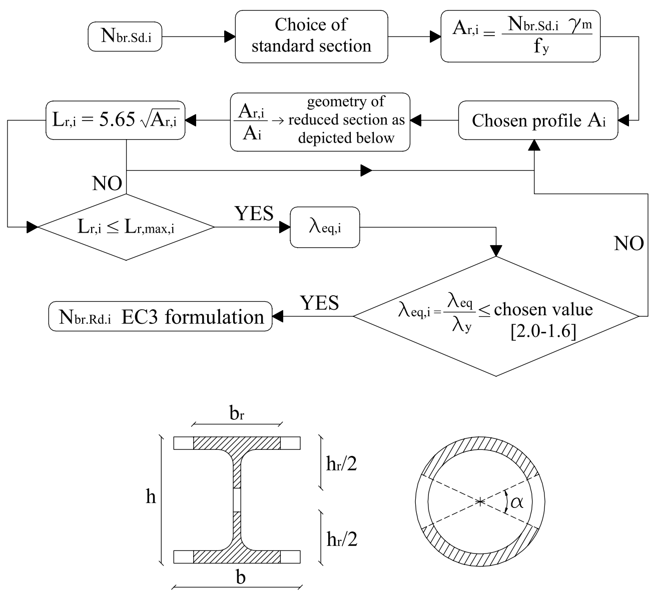

Fig. 3. Design algorithm for bracing members with a RSS.

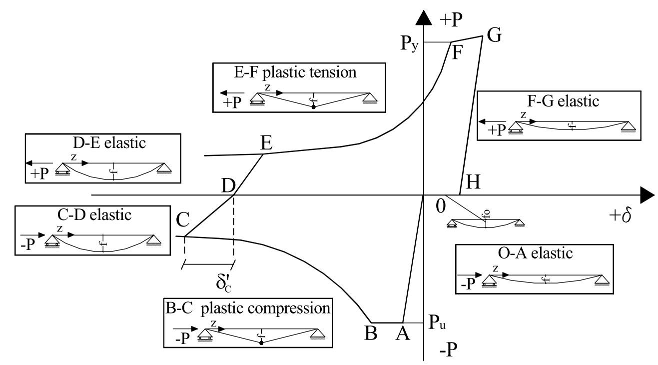

Fig. 4. Theoretical behaviour for a pin-ended brace member.

where $M_{p.r}(N)$ and $M_p(N)$ are the plastic moments of the reduced section and of the gross section, respectively, as affected by the axial force N. By combining Eqs. (14) and (15), the maximum reduced section zone length is obtained:

$$L_{r.\max} = \frac{L}{\pi} \cdot \arcsin \frac{M_{p.r}(N)}{M_p(N)}. \tag{16}$$

However, the need to keep as small as possible the influence of the reduced section zone on the overall slenderness of brace members suggests the use of the lower bound $L_{r,\min}$.

It is important to underline that, according to the above design criteria of bracing members with a RSS, under tensile actions yielding occurs at the brace ends, where the reduced section zones are located and, in addition, the straightening of the brace, which previously buckled in compression, is also developed. Conversely, under compression the behaviour of the RSS bracing members is equal to the one of traditional braces, because it is governed by the midspan section. In addition, it can be recognized that the aim of the RSS bracing members is not an improvement of the energy dissipation capacity, with respect to the traditional braces, because buckling still occurs and is governed by the midspan section's behaviour, but the aim is the calibration of the yield resistance in tension aiming to reduce the brace overstrength Ω which, otherwise, leads to column oversizing.

The design procedure of bracing members with a RSS is shown in the flow chart depicted in Fig. 3. The first step is the determination of the design values $N_{b.Sd.i}$ of the internal actions in the bracing members under the seismic load combination, by means of structural analysis. Afterwards the profiles can be chosen from standard shapes and the reduced sections can be calibrated so that a value equal to 1.0 can be obtained for the overstrength factor Ω_i of every diagonal brace. In particular, as shown in Fig. 3, the reduced flange length b_r and, eventually, the reduced section height h_r can be determined for I sections, whereas the angle α can be obtained for tubes. Successively, the reduced section zone length is determined by means of Eq. (11) and, if the condition $L_{r,i} \le L_{r,\max,i}$ is satisfied, the overall brace slenderness λ_{eq} can be evaluated by means of Eq. (9). The corresponding normalised slenderness $\overline{\lambda}_{eq} = \lambda_{eq}/\lambda_y$ must be less than or equal to the code specified limit. If the above requirement is not satisfied, the brace section has to be increased. Finally, the buckling design resistance ($N_{br.Rd}$) of the bracing member with a RSS is determined according to Eurocode 3 [10] accounting for the reduced section zone influence by means of the equivalent slenderness λ_{eq}.

Aiming to investigate the seismic performances of concentrically braced frames with diagonals designed by means of the RSS strategy, an appropriate modelling of brace cyclic behaviour has to be carried out. To this end, the Georgescu model can be applied [16] by properly accounting for the influence of the reduced section zones on the parameters governing the cyclic response of braces.

Regarding the cyclic behaviour of bracing members, it is depicted in Fig. 4, with reference to the first cycle, where δ and P represent, respectively, the axial deformation and the axial force. The first branch O–A corresponds to the elastic response in compression up to point A where the buckling resistance load is reached. Regarding point B, it corresponds to the complete development of the flexural plastic resistance of the midspan section, due to second order bending moments arising from lateral deflection. Therefore, a kinematic mechanism develops. Obviously, the O–A–B curve is bilinear provided that plastic redistribution at the section level is neglected. By imposing the yielding condition of the midspan section, the mechanism equilibrium curve corresponding to the branch B–C is obtained, until the beginning of unloading (point C). The branch C–D corresponds to the elastic unloading in compression. The reloading branch in tension D–E is also characterised by a linear behaviour with a progressive reduction of the lateral deformation, until the bending moment in the midspan section, due to the residual plastic lateral deformation, reaches again a yielding value (point E). Starting from point E, a new kinematic mechanism develops. The branch E–F is governed by the mechanism equilibrium curve similarly to the B–C branch, obtained by imposing the yielding condition of midspan section. The final point F corresponds to the attainment of the yielding force in pure tension. Due to strain-hardening effects, the branch F–G exhibits increasing behaviour up to the final point G which corresponds to a new unloading phase. In the following cycles, the same modelling can be applied, but a significant degradation of the buckling load (P_u) is expected, due to both the Baushinger effect and plastic elongations accumulated during the previous loading history. For this reason, in the analyses herein presented, a value equal to 50% of the initial buckling load has been assumed for the successive cycles.

The bracing member designed by means of RSS is characterised by the following axial stiffness:

$$K_{br} = \frac{EA}{L}\left[\frac{1}{1 + 2\frac{L_r}{L}\left(\frac{A}{A_r} - 1\right)}\right] \tag{17}$$

with A_r and L_r representing, respectively, the cross sectional area and the length of the reduced section zones, while A and L are the cross section area and the length of the gross section (Fig. 1).

In addition, it must be considered that both the design buckling resistance and the yielding axial forces have to be properly defined

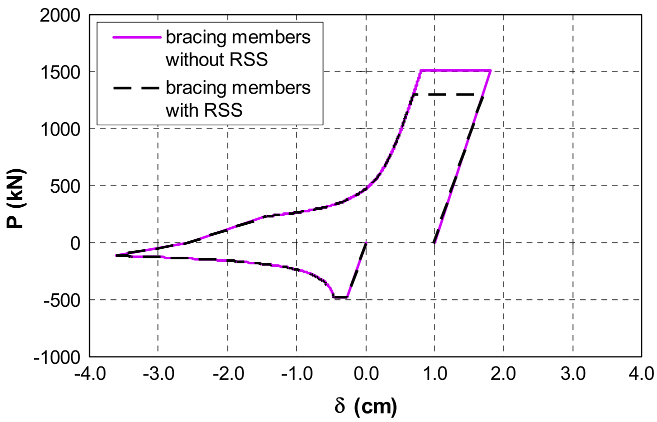

Fig. 5. Comparison between the cyclic response of bracing members with and without a RSS (HEA220, $L = 720$ cm, $A_r = 55.52$ cm^2 and $L_r = 45.0$ cm).

for the members with reduced section zones. While the first one, as already stated, is computed by means of Eurocode 3 [10] accounting for the equivalent slenderness λ_{eq}, the design axial resistance in tension is obtained as:

$$N_{pl.r} = A_r \cdot f_y/\gamma_m. \tag{18}$$

Therefore, accounting for the above parameters defining the brace behaviour, the Georgescu model can be applied to describe the cyclic behaviour of members with a RSS. In Fig. 5 a comparison between the cyclic behaviour of a bracing member (HEA 220, length 720 cm) with and without the RSS (cross section area and length of the reduced section zone, respectively, equal to $A_r = 55.52$ cm^2 and $L_r = 45.0$ cm) is shown with reference to the first cycle. It can be observed that the only important difference is obtained for the yielding axial force in tension, being the remaining parts of the cyclic response practically unaffected by the reduced section zones.

It is useful to note that the available ductility of braces with reduced sections located at their ends is not less than the available ductility of the same brace member without reduced sections. In fact, when the completely straightened diagonal is in tension, only reduced sections are subjected to yielding, while the midspan section is in the elastic range. Conversely, under compression, if the design condition of Eq. (16) is satisfied, the reduced sections remain in elastic range while a plastic hinge develops in the midspan section of the member. Yielding in tension can occur in the midspan section provided that the strength degradation of the midspan section is very significant; this can occur only after several cycles. In a member without a RSS yielding in tension and yielding in compression usually develops in the same section (generally the midspan section) leading to a premature collapse if compared to the same member with RSS. In addition, both experimental analyses and FEM simulations [17,18] confirm that the use of a minimum length for the reduced section zone corresponding to Eq. (11) assures adequate ductility. In fact, a smaller length of the reduced section zone can lead to the spreading of the plastic deformations also in the gross cross section, thus undermining the available ductility.

4. The proposed design methodology

In order to prevent the development of a soft-storey collapse mechanism, the non dissipative elements, i.e. the beams and columns, have to be dimensioned by computing the axial forces transmitted by the dissipative elements when a collapse mechanism of a global type is developed. The proposed methodology

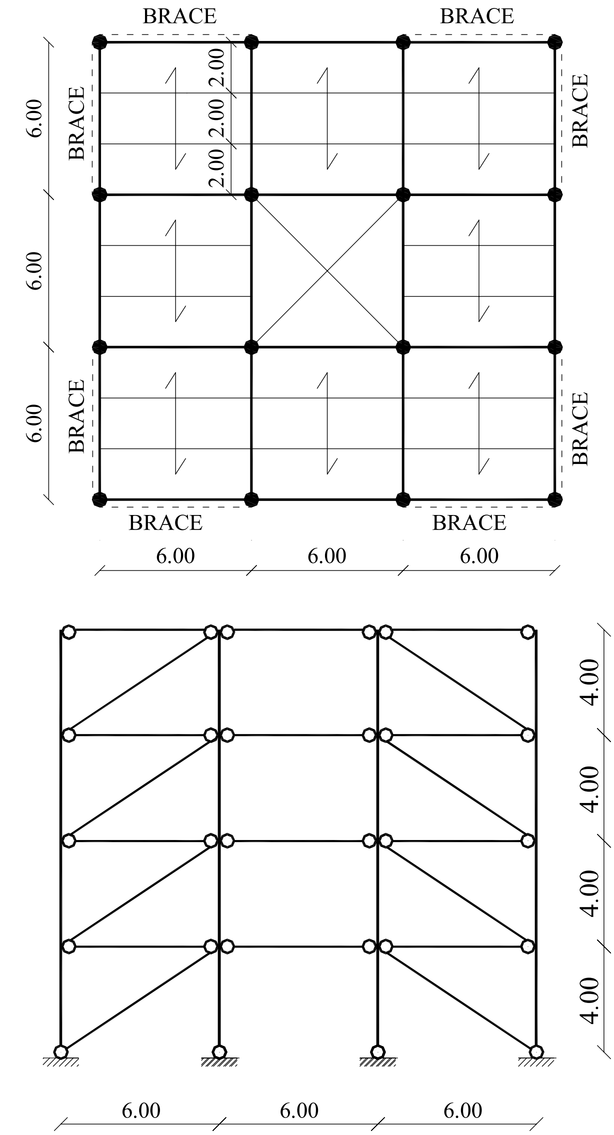

Fig. 6. Plan configuration and structural scheme of analysed buildings.

[6,7] is based on a rigorous application of the "capacity design" philosophy which requires that dissipative zones have to be designed to withstand the internal actions coming from the seismic design horizontal forces. Conversely, the non dissipative zones have to be designed considering the maximum internal actions that the dissipative zones, yielded and strain-hardened, are able to transmit. According to this criterion, the diagonals are designed under the internal actions coming from the seismic horizontal forces (Eq. (1)), by imposing the code limits about normalised slenderness; whereas, the design of beams and columns requires the evaluation of the distribution of internal actions occurring when the collapse mechanism of a global type is completely developed. To this end, the proposed methodology also considers the buckled bracing member's contribution by evaluating their post-buckling behaviour [6,7]. Nevertheless, even though this latter has an important role in X-braced [6] and V-braced frames [7], the evaluation of the compressed member's post-buckling behaviour is not necessary for the analysed structural scheme. In fact, because of the structural scheme herein analysed (Fig. 6), the maximum axial forces in the columns are simply obtained when all the upper diagonals are completely yielded.

In Fig. 7 the deformed configuration of the structure corresponding to the global collapse mechanism is shown. As is possible

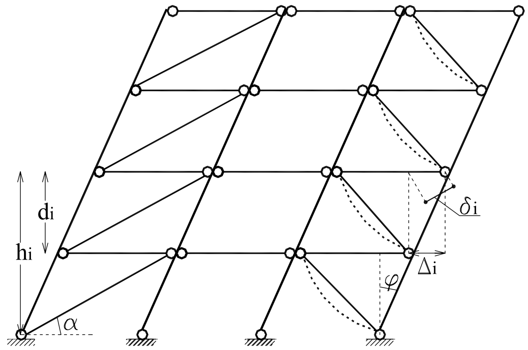

Fig. 7. Structural configuration for a global collapse mechanism.

to observe, the structural scheme is characterised by the continuity of columns; in addition all the beams and the diagonals are end pinned. The maximum axial forces in the beams and columns are given by:

$$N_{b.Sd.E.i} = N_{pl.i} \cdot \cos\alpha_i \tag{19}$$

$$N_{c.Sd.E.i} = \sum_{j=i}^{n} N_{pl.i} \cdot \sin\alpha_i \tag{20}$$

where $N_{pl.i}$ is the design axial resistance of the i-th storey bracing member and α_i is its inclination with respect to the beam. Therefore, the design of the non dissipative members is carried by fulfilling the following relationships:

$$N_{b.Rd.i}(M_{Sd.i}) \geq N_{b.Sd.i} = N_{b.Sd.E.i} + N_{b.Sd.G.i} \tag{21}$$

$$N_{c.Rd.i} \geq N_{c.Sd.i} = N_{c.Sd.E.i} + N_{c.Sd.G.i} \tag{22}$$

where $N_{b.Rd.i}(M_{Sd.i})$ is the in-plane beam design buckling resistance accounting also for the presence of the design bending moment due to the vertical loads included in the seismic load combination and $N_{c.Rd.i}$ is the out of plane buckling resistance of the columns; $N_{b.Sd.E.i}$ and $N_{c.Sd.E.i}$ are the maximum axial forces in the beams and columns which the diagonals are able to transmit in a global type collapse mechanism (given by Eqs. (19) and (20)); $N_{b.Sd.G.i}$ and $N_{c.Sd.G.i}$ are the axial forces in the beams and columns due to gravity loads acting in a seismic load combination. It has to be underlined that, regarding the beams, the out of plane buckling is usually restrained due to the presence of the concrete deck, so that the Eq. (21) applies to the in-plane check; on the contrary, in the case of columns, Eq. (22) has to be applied with reference to both in-plane and out-of-plane buckling.

5. Design example and results of seismic analysis

In order to investigate, from a deterministic point of view, the seismic performances obtained by adopting different design approaches, the four storey building depicted in Fig. 6 has been considered. In the same figure, the structural scheme assumed for the concentrically braced frames is also depicted. All the beam-to-column connections are pinned, therefore the seismic actions are withstood by the concentrically braced frames located along the perimeter of the structure. The values of the dead and live loads are, respectively, equal to 4 kN/m^2 and 2 kN/m^2. The steel grade is S235, so that the value of the yield stress is $f_y = 235$ MPa.

The seismic effects, according to Eurocode 8 provisions, are determined by using a linear elastic model of the structure and the lateral force method of analysis, because of the building's regularity in elevation. The seismic base shear force F_b, for each main horizontal direction in which the structure is analysed, is given by:

$$F_b = S_d(T_1) \cdot \frac{W}{g} \tag{23}$$

Table 1
Storey horizontal seismic forces.

Storey	z_i (m)	W_i (kN)	F_i (kN)
1	4	2267.41	199.68
2	8	2267.41	399.37
3	12	2267.41	599.05
4	16	2171.20	764.84

where T_1 is the fundamental period of vibration of the building in the considered direction, evaluated as $T_1 = 0.05 \cdot H^{3/4}$, being H the total height of the building in metres; $S_d(T_1)$ is the ordinate of design spectrum computed as $S_d(T_1) = S_e(T_1)/q$, being $S_e(T_1)$ the elastic spectral acceleration corresponding to the fundamental period of vibration of the building and q the behaviour factor, taken equal to 4.0 for the concentrically braced frames and the buildings with a regularity in plan and elevation; W is the total seismic weight of the building.

The seismic horizontal forces along the height of the building are determined by means of the following relationship:

$$F_i = F_b \cdot \frac{W_i \cdot z_i}{\sum_j W_j \cdot z_j} \tag{24}$$

where F_i is the seismic horizontal force at the i-th storey; F_b is the seismic base shear force; W_i and W_j are the seismic weights of i-th storey and the j-th storey, respectively, while z_i and z_j are the corresponding heights with respect to the foundation level.

Because of plan's regularity of the analysed building, according to code provisions, it is possible to consider two planar models, one for each main horizontal direction. Therefore, by neglecting accidental torsional effects, each CBF has to withstand the half part of the seismic horizontal forces, determined by Eq. (24), for each storey and for each direction. Table 1 summarises the results obtained applying the aforesaid procedure by assuming soil type A (stiff soil conditions) and a high seismicity zone with a peak ground acceleration equal to 0.35 g.

Starting from the seismic horizontal forces, the design values of the axial forces ($N_{Sd.E.i}$) are obtained by assuming a structural scheme with only tension diagonals active, because the compression ones are assumed to be buckled. While the bracing members are designed considering these axial actions, regarding non dissipative elements (i.e. columns and beams), two design approaches have been considered. The first one corresponds to the simple application of Eurocode 8 [1] provisions, briefly summarised in Section 2, whereas the second approach corresponds to the design methodology described in Section 4. In particular, in order to provide a comparison between the seismic responses obtained using the described design methodologies, four different buildings (i.e. four different CBFs) have been analysed:

- building designed according to Eurocode 8 provisions (in the following briefly named EC8);
- building designed according to the methodology herein proposed (in the following briefly named PROPOSED);
- building designed as the previous point, but by applying the reduced section solution strategy and imposing an upper limit to the normalised slenderness of the diagonals equal to 1.6 (in the following briefly named P-RSS16);
- building designed as the previous point, but imposing an upper limit to the normalised slenderness of the diagonals equal to 2.0 (in the following briefly named P-RSS20).

The last two cases allow one to investigate the influence of the upper limit of the braces' slenderness. In fact, by using a RSS, the limitation of the braces' slenderness can be easily obtained without any problem of oversizing. It is important to underline that the use of an upper limit equal to 1.6, instead of the code's suggested value

Table 2
Standard shapes adopted for CBFs designed according to Eurocode 8 and the proposed methodology.

	Storey	Braces	Ω_i	$\bar{\lambda}_i$	$N_{pl.Rd}$ (kN)	$N_{br.Rd}$ (kN)	Beams	Columns
EC8	1	HEA 220	1.16	1.39	1373.68	483.32	HEA 300	HEB 300
	2	HEA 200	1.09	1.54	1149.36	346.56	HEA 300	HEB 260
	3	HEA 160	1.01	1.93	828.91	172.82	HEA 280	HEB 200
	4	HEA 160	1.80	1.93	828.91	172.82	HEA 260	HEB 140
Proposed	1	HEA 220	1.16	1.39	1373.68	483.32	HEA 300	HEB 360
	2	HEA 200	1.09	1.54	1149.36	346.56	HEA 280	HEB 280
	3	HEA 160	1.01	1.93	828.91	172.82	HEA 260	HEB 220
	4	HEA 160	1.80	1.93	828.91	172.82	HEA 260	HEB 180

Table 3
Standard shapes and geometric properties of braces for CBFs designed according to the proposed methodology including a reduced section solution for braces.

	Storey	Braces	Ω_i	$\bar{\lambda}_i$	b_{red}/b	h_{red}/h	L_{red} (cm)	A_{red} (cm^2)	$N_{pl.r}$ (kN)	$N_{br.Rd}$ (kN)	Beams	Columns
P-RSS16	1	HEA 220	1.00	1.39	0.82	1.00	45.0	55.52	1186.02	482.76	HEA 280	HEB 320
	2	HEA 200	1.00	1.54	0.89	1.00	40.0	49.33	1053.88	346.22	HEA 280	HEB 260
	3	HEA 200	1.00	1.54	0.61	1.00	35.0	38.31	818.44	346.13	HEA 260	HEB 200
	4	HEA 200	1.00	1.54	0.21	0.94	30.0	21.53	460.00	345.97	HEA 240	HEB 140
P-RSS20	1	HEA 220	1.00	1.39	0.82	1.00	45.0	55.52	1186.02	482.73	HEA 280	HEB 320
	2	HEA 200	1.00	1.54	0.89	1.00	40.0	49.33	1053.88	346.20	HEA 280	HEB 260
	3	HEA 160	1.00	1.93	0.98	1.00	35.0	38.31	818.44	172.79	HEA 260	HEB 200
	4	HEA 160	1.00	1.93	0.40	1.00	30.0	21.53	460.00	172.64	HEA 240	HEB 140

equal to 2.0, leads to a reduction of the out of plane deformations of the gusset plates and to the limitation of the sudden impact load occurring at the end of the straightening phase, thus assuring a more effective protection of the connections.

The results of the design procedures are summarised in Table 2 with reference to CBFs designed with traditional braces and in Table 3 with reference to CBFs designed by means of the proposed design methodology including the reduced section solution.

Aiming to compare the seismic performances of the designed CBFs, non linear dynamic analyses have been carried out by means of a PC-ANSR computer program [9]. The bracing members are modelled by using the "non-linear brace element" with pinned ends; the parameters describing their cyclic behaviour have been calibrated in order to match the energy dissipation provided by the Georgescu model [16]. Columns and beams are modelled by using the "non-linear beam column element". The structural model is based on the continuity of columns, while the beams are assumed to be pin-jointed to the columns.

Each structure has been analysed with reference to five historical ground motions (El Centro 19/05/1940 E–W component, PGA = 0.226 g; Kobe 16/01/1980 N–S component, PGA = 0.629 g; Northridge 17/01/1994 N–W component, PGA = 0.842 g; Petrova 15/04/1979 N–S component, PGA = 0.438 g; Tokyo 1956, N–S component, PGA = 0.075 g). Incremental dynamic analyses (IDA) have been carried out for evaluating the structural seismic response under a ground motion of increasing intensity. The analysis has been carried out until structural collapse occurs. In particular, three failure modes have been considered: out of plane buckling of columns, fracture of diagonal braces and excessive storey damage. As a consequence, the structural response has been evaluated with reference to three different parameters, representing three different damage situations:

- the maximum normalised column axial force ($N_{c.Sd}/N_{c.Rd}$) given by the ratio between the maximum axial force occurring in the time history and the out-of-plane buckling resistance, evaluated according to Eurocode 3 provisions [10]. A value equal to 1.0 identifies the collapse of columns due to out-of-plane buckling;
- the maximum normalised cyclic ductility demand of the diagonals ($\mu/\mu_{\lim}$) expressed by the ratio between the maximum cyclic ductility demand and its limit value. The cyclic ductility is defined as the total inelastic deformation in a cycle, i.e. the sum of the one in tension and that in compression. The limit value ($\mu_{\lim}$) has been computed according to the formulation proposed by Tremblay [19]:

$$\mu_{\lim} = 2.4 + 8.3 \cdot \bar{\lambda} \tag{25}$$

  where $\bar{\lambda}$ is the normalised brace slenderness. A value of $\mu/\mu_{\lim}$ equal to 1.0 identifies the collapse of the bracing members due to excessive inelastic deformations leading to fracture;
- the maximum interstorey drift ratio (MIDR), which expresses a measure of storey damage, for both the structural and non-structural components. A limit value of 2% has been assumed as suggested by FEMA 273 recommendations [4] for the limit state of "Collapse Prevention".

It can be reasonably assumed that structural failure occurs when one of the structural response parameters reaches its limit value. It is important to underline that the performed analyses are deterministic, so that the influence of randomness (such as record-to-record variability), uncertainty (such as limited knowledge and modelling of structural system), and also the probabilistic distribution law of capacity are not considered. A more accurate evaluation of seismic structural performances, by examining all these sources of uncertainty, can be carried out by means of a probabilistic analysis providing the structural reliability in terms of mean annual frequency of exceeding a given limit state [20,21].

In Fig. 8 the maximum values of the interstorey drift ratio obtained by IDA analysis, for each designed concentrically braced frame, are depicted with reference to the El Centro record. In addition, the values of the peak ground acceleration leading to collapse (i.e. one of the damage parameters reaches its limit value) are reported. These values correspond to the out-of-plane buckling of columns, in the case of CBF designed according to Eurocode 8, and to the fracture of bracing members in the case of CBFs designed according to the proposed methodology, with and without the RSS strategy.

With reference to Fig. 8, it is also useful to note that the maximum interstorey drift for CBFs designed with the RSS strategy occurs, with reference to ultimate conditions, for structure P-RSS20 for a PGA equal to about 0.38 g. Such maximum interstorey drift is equal to 0.019 corresponding to a brace elongation equal to 6.32 cm. Considering that the length of the reduced section zones is equal to 35 cm (Table 3), the corresponding strain demand is equal to $6.32/(2 \times 35) = 0.09$. This value is compatible

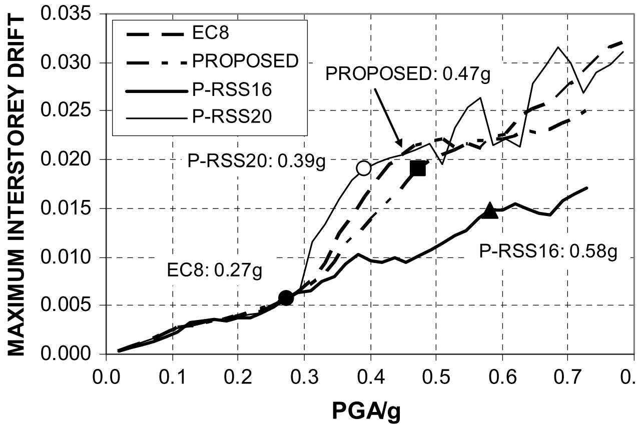

Fig. 8. Maximum interstorey drift versus PGA for each CBF with reference to El Centro record.

with the material's properties, because Eurocodes [1,10] require an elongation at failure, on a gauge length equal to the one provided by Eq. (11), not less that 15%. Therefore, it can be concluded that the strain levels equal to those corresponding to the brace elongation demands occurring in the design examples herein presented can be surely withstood by the common steel grades whose elongation at failure usually exceeds 20%.

In Fig. 9 the maximum ductility demand μ/μ_{lim} of the diagonal members versus the PGA is depicted for the different designed CBFs and for each storey with reference to the El Centro record. The PGA values corresponding to collapse are also pointed out with a vertical line. As can be observed, the CBFs designed according to Eurocode 8 provisions and to the proposed methodology exhibit the most severe plastic engage at the third storey, the one corresponding to the minimum overstrength factor Ω_i. In particular, dealing with the CBF designed according to Eurocode 8 provisions, the value of μ/μ_{lim} at the third storey is less than 1.0, due to the premature buckling of columns at the third storey. Conversely, regarding the CBF designed according to the proposed methodology, the failure is due to the occurrence of fracture of the diagonal braces at the third storey, so that the corresponding damage parameter reaches a value equal to 1.0. A major involvement of the other storeys in the earthquake input energy dissipation can be obtained using the RSS strategy by means of the calibration of diagonal brace sections so that all the overstrength factors, i.e. at each storey, are equal to 1.0. In addition, it can be observed that, even in the case of the CBFs designed according to the proposed methodology with the RSS strategy, the governing failure mode is the fracture of diagonal members independently of the limit value of the normalised slenderness, leading to μ/μ_{lim} values equal to 1.0. In addition, in Figs. 10 and 11 the hysteretic loops of the bracing members of third storey are depicted with reference to the El Centro record and with reference to the EC8 and PROPOSED design methodology, respectively. The captions of the same figures also provide the PGA values corresponding to the collapse. Regarding the above example figures, they have been selected considering that, as mentioned above, the third storey is the one subjected to the most severe plastic engage.

In Fig. 12 the mean values, with respect to the five considered historical ground motions, of the storey dissipated energy and of the total dissipated energy are depicted for the four analysed structures with reference to the mean PGA value leading to collapse, so that the total dissipated energy assumes the meaning of the energy dissipation capacity. Therefore, in the same figure, also the mean value of the PGA leading to collapse is reported. It can be observed that Eurocode 8 design methodology provides the worst behaviour both in terms of PGA values leading to collapse and in terms of energy dissipation capacity. This unsatisfactory behaviour arises because after yielding of bracing members corresponding to the minimum value of the overstrength factor Ω_i (in this case at the third storey, see Table 2), premature buckling of the corresponding columns occurs. In addition, due to the oversizing of upper storey's bracing members, resulting

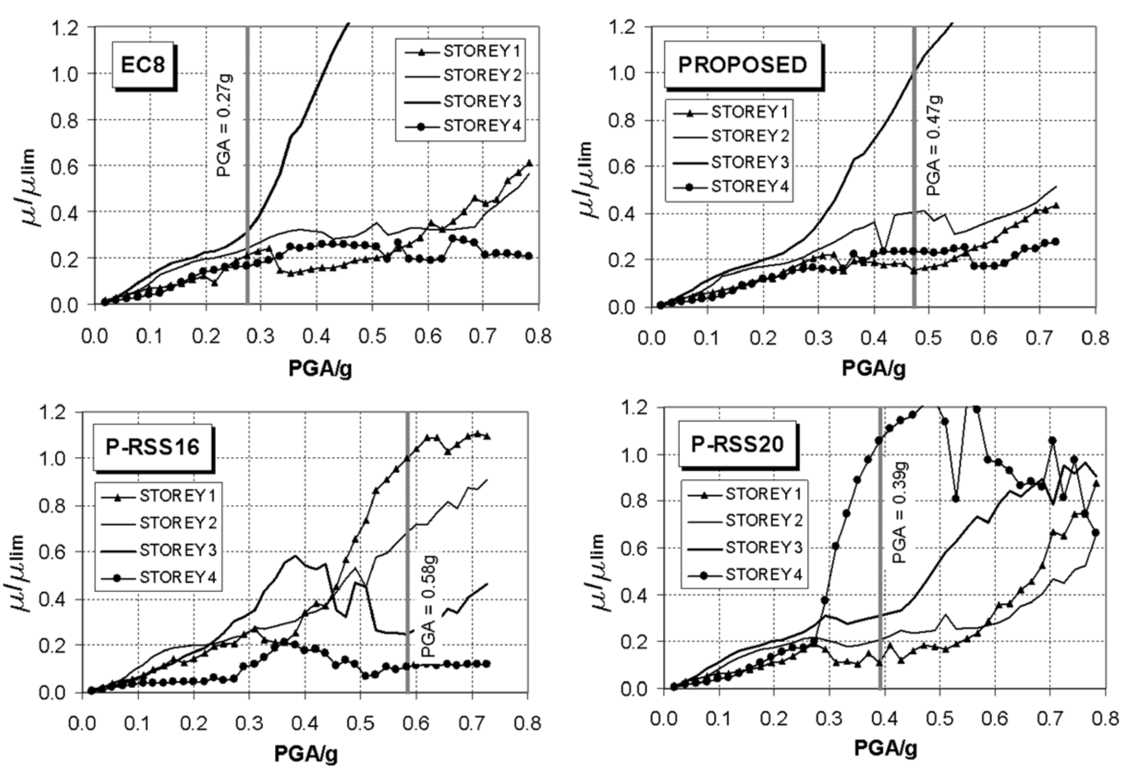

Fig. 9. Maximum non dimensional ductility demand μ/μ_{lim} of diagonal members, versus the PGA for each storey and for the different designed CBFs with reference to the El Centro record.

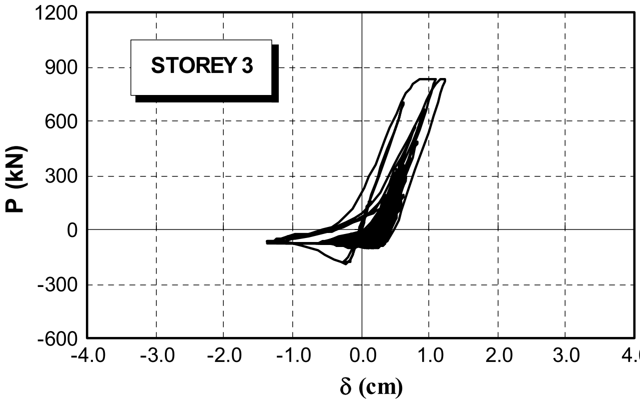

Fig. 10. Brace cyclic response for the El Centro record scaled to PGA = 0.27 g with reference to the CBF designed according to Eurocode 8.

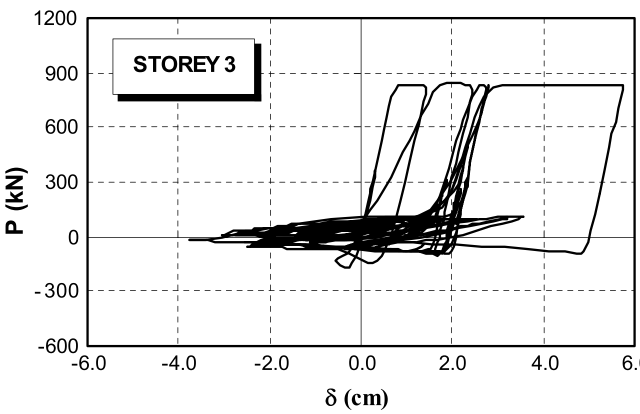

Fig. 11. Brace cyclic response for the El Centro record scaled to PGA = 0.47 g with reference to the CBF designed according to the proposed methodology.

from the normalised slenderness limitation, such storeys exhibit low inelastic deformations providing a poor contribution to the energy dissipation (Fig. 12). A considerable increase of energy dissipation capacity, of about 3.5 times, is reached by means of the proposed design methodology (Fig. 12) where only the top storey's bracing members do not develop a significant plastic behaviour. The limited contribution of the top storey bracing members is, also in this case, due to the normalised slenderness limitation, required by the code, whose application leads to a value Ω_4 significantly greater than the other storeys (Table 2).

A more uniform involvement of all the storeys to the dissipation of the earthquake input energy is obtained by means of the RSS strategy by limiting the normalised slenderness of bracing members to the code suggested value equal to 2.0 (see P-RSS20 in Fig. 12). On the contrary, by reducing the normalised slenderness limitation to 1.6, the energy dissipation capacity increases (see P-RSS16 in Fig. 12), but a less uniform distribution of the storey's dissipated energy is obtained in comparison with P-RSS20. This result can be justified considering that the reduction of the upper bound of the normalised slenderness leads, on one hand to the improvement of the energy dissipation capacity of bracing members, but, on the other hand, it gives rise to a reduction of the limit value of the cyclic ductility ($\mu_{\lim}$) [19], according to Eq. (25).

In Fig. 13 the $N_{c.Sd}/N_{c.Rd}$ ratio versus the PGA curves is depicted, for each storey and for each design methodology, with reference to the El Centro record. It shows that the proposed design methodology, with and without the RSS, always leads to a value of the above ratio of less than 1.0. This is the main goal of the described design methodology, aiming at the safeguard of non dissipative elements preventing the premature buckling of columns. Conversely, the application of Eurocode 8 design provisions does not assure the prevention of columns buckling, the $N_{c.Sd}/N_{c.Rd}$ ratio being greater than 1.0 for low values of the PGA. Finally, the results obtained for the different earthquake records are summarised in Table 4, where the PGA values corresponding to the different failure modes are given. In addition, the governing failure mode is also pointed out.

It is also useful to compare the different design methodologies from the economic point of view. To this end the variation of the constructional steel weight has been evaluated. Assuming as reference the CBF designed according to Eurocode 8 provisions, two parameters can be introduced [22]:

- i_c expresses the CBF's weight influence factor as the ratio between the total weight of concentrically braced frames ($P_{c.\mathrm{EC8}}$) (i.e. the sum of the weight of all the CBFs of the structural system) and that of the whole structure including the leaning part ($P_{s.\mathrm{EC8}}$), both evaluated for the reference design methodology (Eurocode 8):

$$i_c = P_{c.\mathrm{EC8}}/P_{s.\mathrm{EC8}}. \tag{26}$$

- $\Delta P_{c.j}$ is the relative variation of the CBF's total weight, designed according to the j-th design methodology, with respect to Eurocode 8:

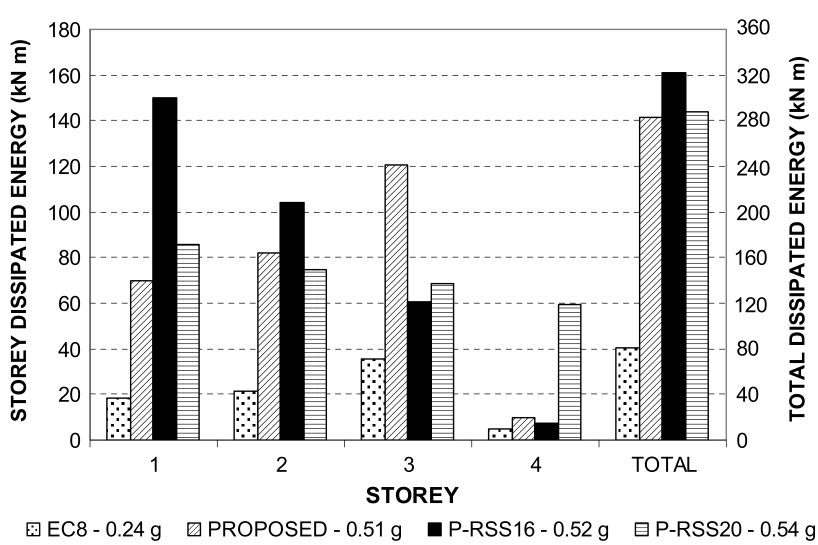

Fig. 12. Mean value of storey and total dissipated energy with reference to the PGA value leading to collapse.

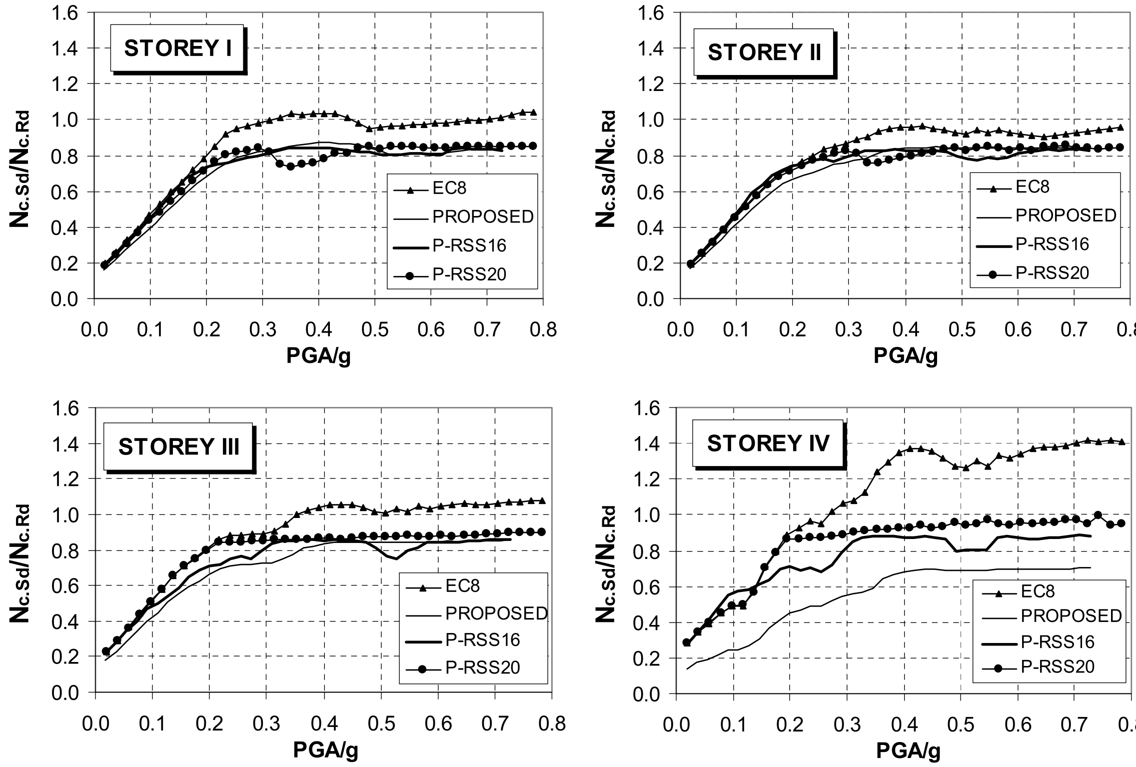

Fig. 13. Maximum non dimensional axial force $N_{c.Sd}/N_{c.Rd}$ in the columns versus PGA for the El Centro record.

Table 4
PGA values corresponding to the different failure modes for the designed structures.

	Record	Excessive storey damage (g)	Fracture of diagonal braces (g)	Out-of-plane buckling (g)	Governing failure mode (g)
EC8	El Centro	0.45	0.43	0.27	0.27
	Kobe	0.67	0.31	0.22	0.22
	Northridge	0.45	0.45	0.27	0.27
	Petrova	0.42	0.42	0.14	0.14
	Tokyo	0.63	0.68	0.30	0.30
				Mean value	0.24
Proposed	El Centro	0.49	0.47	N.B.	0.47
	Kobe	0.72	0.46	N.B.	0.46
	Northridge	0.44	0.46	N.B.	0.44
	Petrova	0.43	0.46	N.B.	0.43
	Tokyo	0.72	0.83	N.B.	0.72
				Mean value	0.50
P-RSS16	El Centro	N.R.	0.58	N.B.	0.58
	Kobe	0.67	0.43	N.B.	0.43
	Northridge	0.37	0.39	N.B.	0.37
	Petrova	0.49	0.51	N.B.	0.49
	Tokyo	0.72	0.75	N.B.	0.72
				Mean value	0.52
P-RSS20	El Centro	0.43	0.39	N.B.	0.39
	Kobe	0.77	0.72	N.B.	0.72
	Northridge	0.41	0.43	N.B.	0.41
	Petrova	0.34	0.45	N.B.	0.34
	Tokyo	0.82	0.90	N.B.	0.82
				Mean value	0.54

N.B. = no buckling.
N.R. = drift limit not exceeded.

$$\Delta P_{c.j} = \frac{P_{c.j} - P_{c.\mathrm{EC8}}}{P_{c.\mathrm{EC8}}} \tag{27}$$

where $P_{c.j}$ is the total weight of the CBFs designed according to the j-th design methodology.

By combining Eqs. (26) and (27), the following relationship for the CBF's total weight, with reference to the j-th design methodology, can be easily derived:

$$P_{c.j} = (1 + \Delta P_{c.j}) \cdot P_{c.\mathrm{EC8}} = (1 + \Delta P_{c.j}) \cdot i_c \cdot P_{s.\mathrm{EC8}}. \tag{28}$$

The whole structural weight corresponding to the j-th design methodology $P_{s.j}$ is obtained by the sum of CBF's total weight $P_{c.j}$ and the weight of the part of the structure P_v resisting vertical loads only (leaning part). The weight P_v is not affected by the

Table 5
Building weight and percentage variation.

	$P_{c.j}$ (kN)	$P_{s.j}$ (kN)	$\Delta P_{c.j}$ (%)	$\Delta P_{s.j}$ (%)
EC8	128.76	595.26	–	–
Proposed	135.18	610.88	3.35	2.62
P-RSS16	126.61	585.06	−2.19	−1.71
P-RSS20	123.18	571.33	−5.13	−4.02

design methodology adopted for the seismic resistant part and is expressed as:

$$P_v = P_{s.\mathrm{EC8}} - P_{c.\mathrm{EC8}} = P_{s.\mathrm{EC8}} \cdot (1 - i_c). \tag{29}$$

Therefore, the whole weight of the structure designed according to the j-th design methodology can be expressed as:

$$P_{s.j} = P_{c.j} + P_v = P_{s.\mathrm{EC8}} \cdot (1 + i_c \cdot \Delta P_{c.j}). \tag{30}$$

As a consequence, the variation of the total structural weight, with respect to the one resulting from Eurocode 8 provisions, can be expressed as:

$$\Delta P_{s.j} = \frac{P_{s.j} - P_{s.\mathrm{EC8}}}{P_{s.\mathrm{EC8}}} = i_c \cdot \Delta P_{c.j}. \tag{31}$$

The results obtained by means of Eqs. (27), (28), (30) and (31) are summarised in Table 5 for the analysed structures.

It can be observed that the proposed design methodology, aiming at the safeguard of non-dissipative elements by means of a rigorous application of capacity design principles, provides an increase in structural weight and, as a consequence, in building cost, compared to the structure designed according to Eurocode 8 provisions. Nevertheless these variations are very small, only 2.6%, and completely justified by a significant improvement of the structural seismic performance. In addition, the combination of the proposed design methodology with the reduced section solution leads to a reduction of the structural weight, and as consequence of the building cost, also allowing a significant improvement of the building's seismic performance. This is due to the possibility of calibrating the resistance of diagonal braces to obtain the diagonal's overstrength factors Ω equal to 1.0 at all the storeys, so that in this case the proposed design methodology allows to prevent premature collapse of non dissipative elements without oversizing them. The obtained reduction of the structural weight obviously decreases by assuming a lower limit value for the maximum normalised slenderness of bracing members.

6. Conclusions and future developments

The results of dynamic inelastic analyses of braced frames designed according to different approaches allow one to derive some preliminary conclusions about the effectiveness of the investigated design approaches and about the influence of the limitation concerning the normalised slenderness of braces.

The application of Eurocode 8 provisions leads to structures where failure prematurely occurs due to the columns buckling. The cause of such unsatisfactory seismic behaviour is the underestimation of the column's axial load so that yielding of all the diagonals cannot be attained. As a consequence, the participation to the dissipation of the earthquake input energy is assured only with reference to the storey exhibiting the minimum Ω_i overstrength coefficient.

Conversely, the application of the proposed design methodology provides structures with a good ability to develop plastic excursions, engaging all the storeys. In addition, the use of the Reduced Section Solution combined with the proposed design methodology can lead to a further improvement of the seismic behaviour. In this case, the use of the RSS allows savings in the constructional steel's weight whose magnitude depends on the limiting value of the non dimensional slenderness of braces. The greatest saving in the constructional steel's weight is obtained when the code suggested limitation, equal to 2.0, to the brace's non dimensional slenderness is adopted. Conversely, a less severe limitation, such as 1.6, leads to a minor saving in the constructional steel's weight and, in addition, the dissipation along the height of the structure is less uniform. However, also in this case, the seismic behaviour remains very satisfactory. On the other hand, a more stringent limitation of the brace's slenderness guarantees a better safeguard of connections.

Even though the preliminary performance assessment of the designed buildings is based on IDA analyses limited to five records, the obtained results are encouraging about the improvements in performance, with respect to Eurocode 8 approach, which can be attained by means of the proposed design procedure both with and without RSS.

However, it has to be recognised that the seismic response of structures is highly affected by the frequency content of the ground's motion, so that record-to-record variability should be considered by means of a robust approach. Therefore, the forthcoming development of the present work could be represented by the use of a probabilistic approach leading to a seismic performance assessment in terms of the mean annual frequency of exceeding the specified limit states.

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