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Prediction of the Seismic Response of Steel Frames with Concentric Diagonal Bracings

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Abstract: Concentrically braced frames and eccentrically braced frames are efficient seismic resistant systems but they are generally prone to develop storey collapse mechanisms. As underlined in some previous papers with regard to eccentrically braced frames, this tendency depends on the overstrength factors and damage distribution capacity factors resulting from the use of common design procedures. In this paper a previously proposed procedure which predicts the heightwise distribution of the damage at collapse of eccentrically braced frames is applied to concentrically braced frames. To apply this procedure, proper definitions of the overstrength factors and damage distribution capacity factors are derived. The effectiveness of the proposed procedure is tested on frames with concentric diagonal bracings characterised by different storey numbers and designed by common design procedures. The target response is provided by nonlinear dynamic analyses. The seismic input is constituted by ten artificial accelerograms. The paper proves that the proposed procedure is able to predict accurately the nonlinear dynamic response of systems in which the damage is mainly restricted to a few storeys. In the other cases, some not negligible scattering between actual and expected values of damage can be found at some storeys.

Keywords: Approximated method, concentrically braced frames, damage distribution capacity factor, dual braced systems, overstrength factor, seismic response.

INTRODUCTION

Eccentrically braced frames and concentrically braced frames endowed with either traditional or buckling restrained braces are generally characterised by low lateral post-elastic stiffness. Several research studies have shown that these systems, even though designed in compliance with capacity design principles, can show a tendency to a soft storey collapse mechanism [1-8] and that this behaviour is accentuated by a non-homogeneous distribution of the overstrength of dissipative members [9-11]. In traditional concentrically braced frames, a concentration of inelastic deformations at a few storeys is further promoted by the decrease of the brace resistance in the post-buckling range of behaviour and by the reduction of the brace dissipative capacity under cyclic loading conditions [12, 13]. Some researchers have demonstrated, however, that the presence of continuous columns can reduce maximum and residual drift demands [14, 15] and cause a more uniform distribution of the drift demand along the height of the building [6, 16-21].

To achieve a global dissipative behaviour of the structure, Eurocode 8 (EC8) [22] stipulates that the maximum value of the overstrength should not exceed the minimum value by more than 25% (*homogeneity strength condition of dissipative members*). Past studies have demonstrated, however, that eccentrically and concentrically braced frames may develop a storey collapse mechanism even if the homogeneity strength condition is verified in design [23]. Further, it has been demonstrated that the seismic response at collapse of traditional and tied eccentrically braced frames [24] is related to the overstrength factor and also to another parameter called “damage distribution capacity factor” in reference [25]. An analytical formulation of the damage of link beams has been proposed as a function of both the storey overstrength and damage distribution capacity factor (*DDC*) to predict the heightwise distribution of the damage at collapse of eccentrically braced structures and to design dual eccentrically braced frames [26, 27]. The *DDC* factor is complementary to the overstrength factor because it is calculated on the basis of the deformation capacity of the dissipative member and on the seismic behaviour of links in the nonlinear range of behaviour. The procedure can be applied to systems that are not characterised by a significant torsional behaviour [28-31].

In this paper a proper definition of the brace overstrength and damage distribution capacity factors is provided with reference to frames with concentric diagonal bracings. Further, the relation proposed with reference to eccentrically braced frames to predict the damage at collapse is extended to simple and dual concentrically braced frames.

PREDICTION OF DAMAGE INDEX

To quantify the heightwise distribution of the damage of braced frames at failure, a storey damage index is defined by means of the following relation

$$DI = \frac{\mu_d - 1}{\mu_f - 1} \qquad (1)$$

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where μ_d is the storey ductility demand and μ_f is the ductility demand corresponding to failure of the storey under examination, i.e. the ultimate storey ductility. This index ranges from 0 to 1. In particular, it is equal to one at the storey where the ductility demand is equal to the ultimate ductility. The mean value of the damage index along the height of the building is indicated later as DI_m. It provides a measure of the propensity of the frame to develop a storey collapse mechanism. Values of DI_m close to unity are obtained in frames where the ultimate ductility is achieved almost simultaneously at all storeys; such values are representative of a ductile collapse mechanism. On the contrary, values of DI_m close to zero are obtained in frames where the damage is localised to a few storeys.

Equation (1) could be used to evaluate the heightwise distribution of damage once the ductility demands in a frame at failure have been calculated by means of nonlinear dynamic analysis. However, this method of analysis entails expertise in correctly modelling the seismic input and the nonlinear cyclic behaviour of structural members and requires a huge computational effort [32]. Owing to this, it is not recommended for every day design use. Based on this consideration, Bosco and Rossi [25] recently formulated a simple procedure for the prediction of the damage of eccentrically braced frames and proposed the following relation for the evaluation of the storey damage index DI

$$\mathrm{DI}_i^e = D_i^{\exp(-3DDC_{\min})} \quad (2)$$

In this relation, DDC_{min} is the minimum value of the damage distribution capacity factor within the structure and D is the storey damage variable; specifically, this latter parameter was calculated as a function of both the overstrength factor Ω_s and the damage distribution capacity factor DDC by means of the following relation

$$D_i = \frac{\min\left\lfloor \Omega_s^5 (1+DDC)^5 \right\rfloor}{\Omega_{s,i}^5 (1+DDC_i)^5} \quad (3)$$

More details regarding the evaluation of the overstrength factor Ω_s and damage distribution capacity factor DDC of eccentrically braced frames can be found in [25].

In the present paper, to ensure Equations (2) and (3) are suitable for the prediction of the heightwise distribution of the damage index of steel frames with concentric diagonal bracings, proper relations and criteria are determined for the evaluation of Ω_s and DDC factor of this structural type.

STOREY OVERSTRENGTH OF BRACED FRAMES

The storey overstrength Ω_s identifies the storey where the premature yielding of the tension brace occurs. In this paper, Ω_s is defined as the ratio of the actual lateral storey strength (V_{Rd}) to that required in design (V_{Ed}). In order to evaluate the storey overstrength factor of frames with concentric diagonal bracings, the lateral storey strength V_{Rd} of this type of frame must be properly defined. To this end, two single-storey diagonal braced frames with different normalised slenderness $\bar{\lambda}$ are subjected to an increasing horizontal top displacement. The normalised slenderness $\bar{\lambda}$ is given by the following formula

$$\bar{\lambda} = \frac{\lambda}{\pi}\sqrt{\frac{f_y}{E}} \quad (4)$$

where λ is the brace slenderness ratio, E and f_y are the modulus of elasticity and the yield stress of steel, respectively. Specifically, values of the normalised slenderness equal to 1.3 and 2.0 are selected because they represent the minimum and maximum normalised slenderness allowed by EC8 for diagonal braces [22, 33]. Independently of the slenderness of the brace, the base shear - top displacement relation is linear until the axial force of the brace in compression is lower than its buckling resistance $N_{b,Rd}$ (Fig. **1**). The storey shear corresponding to the buckling of the brace in compression is named $V_{b,Rd}$ and is obtained by the expression

$$V_{b,Rd} = 2\left(N_{b,Rd} - \left|N_q\right|\right)\cos\theta \quad (5)$$

where N_q is the axial force due to the gravity loads and θ is the angle of inclination of the brace with respect to the horizontal axis. When the top displacement is increased, the contribution to the horizontal reaction given by the brace in tension increases while the contribution provided by the brace in compression decreases. The increase in the sum of the horizontal components of the axial forces in the braces is possible because the increase in the axial force of the brace in tension is larger than the decrease in the axial force of the brace in compression. The maximum storey shear (V_{max}) is attained when the brace in tension yields. For this value of the maximum storey shear, the brace in compression is subjected to an axial force which is lower than the buckling resistance and higher than the post-buckling resistance $N_{u,Rd}$. When the top displacement is increased up to the displacement u^{lim} corresponding to the ultimate ductility of the braces, the sum of the horizontal components of the axial forces in the braces decreases because the reduction of the axial force of the member in compression is not balanced by the increase of the axial force of the brace in tension. The storey shear corresponding to the ultimate strength is named $V_{u,Rd}$ and is evaluated by means of the relation

$$V_{u,Rd} = \left(N_{pl,Rd} + N_{u,Rd}\right)\cos\theta \quad (6)$$

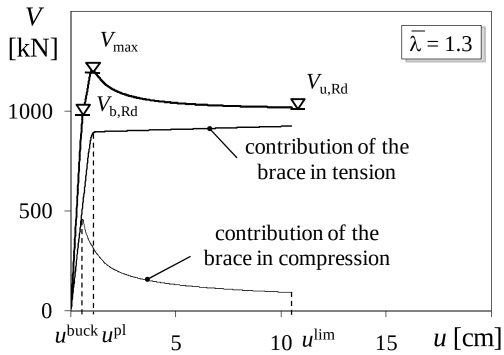
where $N_{pl,Rd}$ is the yield strength of the brace.

In this paper the lateral storey strength V_{Rd} is assumed equal to the ultimate shear force $V_{u,Rd}$ sustained by the braces. This assumption underestimates the maximum storey shear V_{max} by a maximum of 20% (for $\bar{\lambda}$ = 1.3) and is consistent with the design method stipulated by the Japanese seismic code [34-36] or that proposed by Marino and Nakashima [37-39]. Further, it is also consistent in principle with EC8 [22]. This code considers the lateral strength V_{Rd} equal to the shear force corresponding to the yielding of the brace in tension and to a null axial force in the brace in compression. Once the storey lateral strength V_{Rd} has been defined, the storey overstrength can be evaluated as follows

$$\Omega_s = \frac{V_{u,Rd}}{V_{Ed}} \quad (7)$$

DAMAGE DISTRIBUTION CAPACITY FACTOR

The damage distribution capacity factor DDC evaluates the tendency of concentrically braced structures to experience concentration of damage on a few storeys if the yielding of a brace occurs prematurely. In particular, the DDC factor is calculated supposing that the braces in tension of



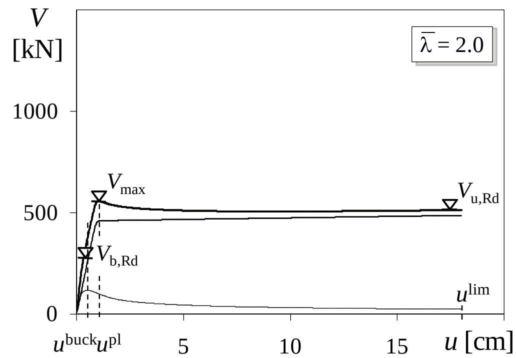


Fig. (1). Shear force – displacement relationship for diagonal braces with normalised slenderness equal to 1.3 and 2.0.

the single storey have yielded while the braces of the other storeys are within the elastic range of behaviour.

To evaluate the *DDC* factor of the *i*-th storey, a modal response spectrum analysis is carried out on the structural model in which the braces of the *i*-th storey are not included so as to simulate the yielding of the brace in tension and the buckling of the brace in compression. On the contrary, an elastic behaviour is assumed for the braces of all the other storeys. However, a reduced stiffness is considered for these braces in order to simulate the extent of stiffness degradation occurring after braces in compression have buckled. The interstorey displacement demands of all storeys are computed and normalised to the ultimate values of the plastic interstorey displacements of the same storeys, i.e. to the plastic interstorey displacements corresponding to the brace failure. Normalisation is carried out with the aim of evaluating the importance of the interstorey displacement demands with respect to the plastic deformative capacity of the same storeys. This is analytically expressed by the relation

$$DDC_{\mathrm{i}} = \frac{1}{n_{\mathrm{s}}-1}\left(\sum_{\mathrm{j=1}}^{\mathrm{ns}} \frac{\Delta u_{\mathrm{j}}}{\Delta u_{\mathrm{j}}^{\mathrm{lim}}}\right)_{\mathrm{j\neq i}} \Bigg/ \left(\frac{\Delta u_{\mathrm{i}}}{\Delta u_{\mathrm{i}}^{\mathrm{lim}}}\right) \tag{8}$$

where n_{s} is the number of storeys, Δu is the required interstorey displacement and Δu^{lim} is the ultimate plastic interstorey displacement, i.e. the difference between the interstorey displacement corresponding to the ultimate ductility of the braces (u^{lim}) and the interstorey displacement corresponding to the end of the elastic behaviour (u^{pl}). All these displacements slightly depend on the distribution of forces applied to the structures.

Definition of the Ultimate Ductility of the Braces

The available ductility μ_{f} of the braces is largely related to the fracture of the cross-section following local buckling, provided that brace connections are adequately designed and detailed [40-42]. This parameter is evaluated here by means of the relation proposed by Tremblay [43]

$$\mu_{\mathrm{f}} = 2.4 + 8.3\bar{\lambda} \tag{9}$$

Equation (9) was determined by Tremblay on the basis of the experimental results of cyclic loading tests; in particular, the ductility of the brace was calculated at each cycle as the sum of the absolute values of the shortening and elongation of the brace (δ^- and δ^+) divided by the axial elongation of the brace at yield δ_{y}.

Evaluation of the Ultimate Plastic Interstorey Displacement

The interstorey displacements corresponding to the ultimate ductility of the braces are calculated here by adding the contributions relative to three different ranges of behaviour of the braced frame. The procedure is applied to braced frames in which only braces are allowed to enter the inelastic range of behaviour. The abovementioned displacement contributions are evaluated by elastic structural models which simulate the mechanical properties of the frame within the range of behaviour under consideration.

The first range of behaviour of the braced frame is characterised by axial forces of braces not higher than the buckling resistances. Consequently, in the first model the braces of all storeys are elastic. The structure is subjected to static lateral forces $F_{\mathrm{i}}^{(1)}$ which, taking into account also the effects of gravity loads, are able to cause the braces of all storeys to buckle (Fig. **2a**) and thus lead the frame to the beginning of the successive range of behaviour

$$F_{\mathrm{i}}^{(1)} = V_{\mathrm{b,Rd,i}} - \sum_{\mathrm{j=i+1}}^{\mathrm{ns}} F_{\mathrm{j}}^{(1)} \tag{10}$$

At the end of the described elastic range of behaviour the interstorey displacement is u^{buck}; the axial elongation of the braces in tension is equal but opposite in sign to the axial shortening of the braces in compression and is indicated as $\delta_{\mathrm{b}}^{(1)}$.

The successive range of behaviour of the frame is characterised by elastic braces in tension and by braces in compression where buckling has already been achieved. The model simulating this second range of behaviour of the structure considers braces in tension but not braces in compression. The static lateral forces $F_{\mathrm{i}}^{(2)}$ lead to an increase of the axial force in the braces in tension equal to $\Delta N_{\mathrm{pl,Rd}} = N_{\mathrm{pl,Rd}} - (N_{\mathrm{b,Rd}} - |N_{\mathrm{q}}|)$ and to a reduction of the axial force in the braces in compression equal to $\Delta N_{\mathrm{u,Rd}} = N_{\mathrm{u,Rd}} - (-N_{\mathrm{b,Rd}} + |N_{\mathrm{q}}|)$ (Fig. **2b**). These lateral forces are given by the following relation

$$F_i^{(2)} = \left(V_{u,Rd,i} - V_{b,Rd,i}\right) - \sum_{j=i+1}^{ns} F_j^{(2)} \qquad (11)$$

The reduction $\Delta N_{u,Rd}$ of the axial force in the braces in compression is not observed in the static analysis of the structures because the braces in compression are not modelled in this stage of the structural behaviour. Owing to this, these forces are applied to the structure at the intersection of braces with columns. The interstorey displacements achieved at the end of this range of behaviour are added to u^{buck} to evaluate those corresponding to the end of the elastic behaviour (u^{pl}). The additional axial elongation and shortening of the braces obtained in this range of behaviour are indicated as $\delta_b^{(2)}$.

Finally, in the third range of behaviour (Fig. **2c**) no increment of the axial forces in braces is possible because the braces in tension are yielded and those in compression are in the post-buckling range of behaviour. The storey stiffness is only provided by the flexural stiffness of the columns but this contribution is neglected. The interstorey displacement can increase until the maximum shortening in the brace δ_{max} (which is assumed to be equal to the maximum elongation) is

$$\delta_{max} = 0.5\,\delta_y\,\mu_f \qquad (12)$$

Thus, the interstorey displacement corresponding to the ultimate ductility of the brace is

$$u^{lim} = \left(\delta_{max} - \delta_b^{(1)} - \delta_b^{(2)}\right)/\cos\theta + u^{pl} \qquad (13)$$

and the ultimate plastic interstorey displacement is:

$$\Delta u^{lim} = u^{lim} - u^{pl} = \left(\delta_{max} - \delta_b^{(1)} - \delta_b^{(2)}\right)/\cos\theta \qquad (14)$$

Evaluation of the Interstorey Displacement Demand

The interstorey displacements Δu which appear in Equation (8) are evaluated by means of a modal response spectrum analysis. The structural model includes the concentrically braced frame as well as the columns designed for gravity loads (or the moment resisting frames in the case of dual systems). The braces of the *i*-th storey (i.e. where the *DDC* factor is calculated) are not included in the numerical model. In fact, at this storey, the brace in compression is considered to have buckled and that in tension is considered to have yielded. The braces of all the other storeys are in the elastic range of behaviour. However, the area $A_{eq,j}$ of the cross section of these braces is lower than the nominal area $A_{br,j}$ to take into account that the lateral storey stiffness reduces from that of the first range of behaviour to that of the second range (Fig. **2d**) before the attainment of the interstorey displacement u^{pl}. The equivalent area of the brace cross-section $A_{eq,j}$ is defined so that the lateral stiffness of the *j*-th storey equals the slope of the elastic branch of the bilinear idealization of the monotonic behaviour represented in (Fig. **2d**). Based on this assumption, the equivalent area of the brace cross-section $A_{eq,j}$ can be calculated by the following relation

$$A_{eq,j} = A_{br,j}\,\frac{V_{u,Rd,j}}{2V_{u,Rd,j} - V_{b,Rd,j}} \qquad (15)$$

ANALYSED STRUCTURES

A set of building structures with concentric diagonal braces is analysed in this paper. The structure is defined by the intersection of four five-span plane frames oriented along the *x*-direction and six three-span plane frames oriented along the *y*-direction. The plan layout (Fig. **3**) as well as the geometric and mass properties are equal at all floors. The length *L* of each span is equal to 8 m while the interstorey height *h* is equal to 3.3 m. The diagonal braces are located along the perimeter of the building. Characteristic values of dead and live loads are equal to 4.4 and 2.0 kN/m², respectively. Storey mass is calculated on the basis of a mean value of the gravity loads equal to 5.0 kN/m² [44]. The buildings

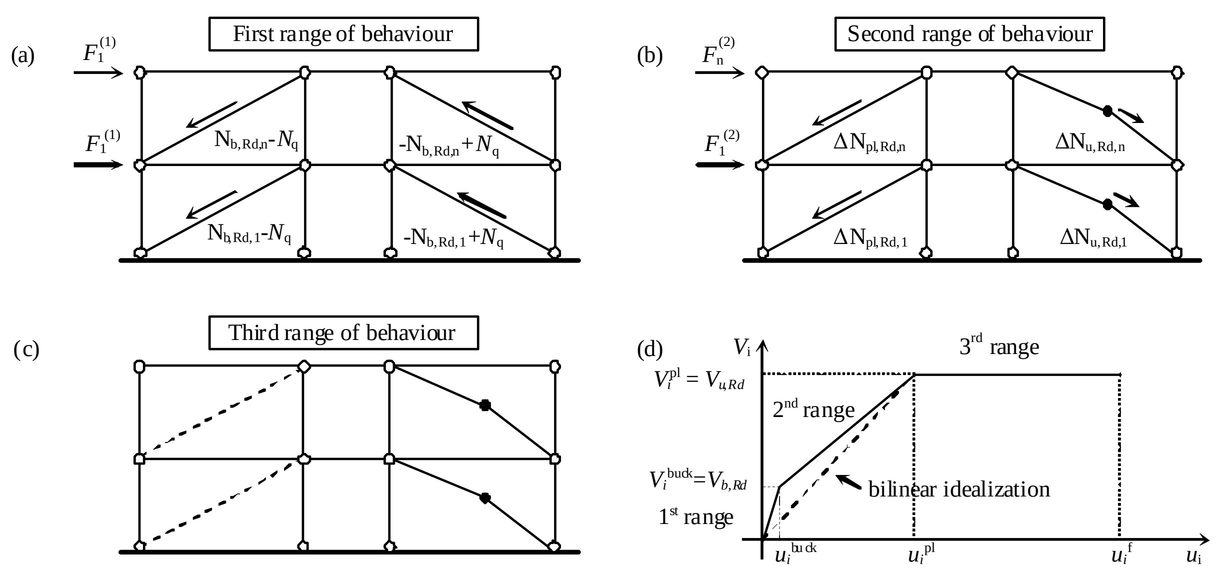


Fig. (2). Idealised monotonic behaviour of the braced frame: (**a**) elastic behaviour, (**b**) behaviour after the buckling of the compression braces, (**c**) behaviour after the yielding of the tension braces, (**d**) storey shear force – interstorey displacement relationship.

are founded on soft soil (soil C according to EC8) in a highly seismic area with a reference ground acceleration $a_{g,R}$ equal to 0.35 g.

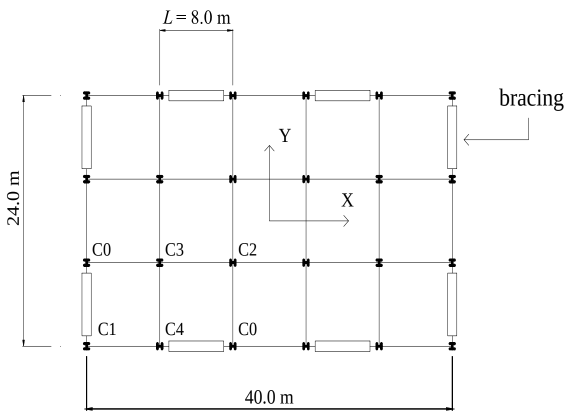


Fig. (3). Plan layout of the considered structures.

The number of storeys of the building n_s is varied from 4 to 12 (in steps of 4). Two seismic-resistant configurations are considered for each structure: the seismic action is resisted only by the braced frames in the first configuration while it is sustained by the braced frames and moment resisting frames (MRFs) in the second. These two configurations will be hereinafter referred to as *simple concentrically braced system* and *dual system*, respectively. The columns are continuous along the height of the building. Beam-to-column connections and column-base connections are always pinned in the plane of the braced frame. All the other connections are assumed to be rigid and full strength in dual systems and pinned in simple concentrically braced systems. For each of the obtained structures, braces are designed according to two different design methods as stipulated in EC8 [22] and proposed by Marino and Nakashima [37-39]. Furthermore, the effect of the seismic force on dissipative members (braces and also beams in the case of dual systems) is evaluated by either the modal response spectrum analysis or the lateral force method of analysis. The total number of frames analysed in this paper is 24 (3 heights x 2 seismic-resistant configurations x 2 design methods of braces x 2 methods of analysis). Each frame is identified by the number of storeys (04, 08 or 12), the design procedure (*EC* for Eurocode 8 or *MN* for Marino and Nakashima) and the method of analysis (*m* for modal response spectrum analysis or *s* for lateral force method of analysis). The symbol *D* is added at the beginning of the label of the frame to indicate dual systems.

The design seismic action is determined by the elastic spectrum proposed by EC8 (1993) reduced by the behaviour factor q. The spectrum is representative of soil type C and characterised by $a_{g,R} = 0.35\ g$. The value of q is assumed equal to 4.0 for braces designed according to EC8, while it is 3.5 when the design method by Marino and Nakashima is used. These values are adopted for both the simple concentrically braced systems and the dual systems. Because of the symmetry of the structure, the numerical analyses for the evaluation of the internal forces of the dissipative members are carried out using a two-dimensional model which represents half the structure of the building. This numerical model includes only the braced frame in case of simple concentrically braced systems and both the braced frame and moment resisting frames for dual systems. Diagonal braces are generally slender and are expected to buckle at a low level of seismic force. Thus, according to a simplification allowed by EC8, only the braces in tension are included in the numerical model adopted for the evaluation of the design internal forces.

Braces are made of square hollow sections and steel grade S235. Wide-flange shapes and steel grades S235, S275 and S355 are used for beams and columns. The same section is used for the columns of two consecutive storeys.

Simple Concentrically Braced Systems

Braces are designed by the method suggested in EC8 [22] and by that proposed by Marino and Nakashima [37-39]. The cross-section size of the braces is determined at each storey by equating the design storey shear force to the shear strength. According to EC8, the lateral strength provided by a pair of diagonal braces is assumed to equal the horizontal reaction of the tension brace, which is supposed to carry the axial force corresponding to its yielding $N_{pl,Rd}$. Therefore, the area of the cross-section of the braces of the i-th storey is

$$A_{br,\,i} = \frac{V_{Ed,\,i}}{f_y \cos\theta} \tag{16}$$

According to Marino and Nakashima [37-39], the shear strength provided by a pair of diagonal braces is equal to the shear $V_{u,Rd}$ calculated by Equation (6) and the area of the cross-section of the braces of the i-th storey is

$$A_{br,\,i} = \frac{V_{Ed,\,i}}{(1+\chi_u)\, f_y \cos\theta} \tag{17}$$

where χ_u is the ratio of $N_{u,Rd}$ to $N_{pl,Rd}$. In this paper, χ_u is evaluated by an equation of the normalized slenderness $\bar{\lambda}$. This equation has been numerically determined and is reported in [10]. Regardless of the design method adopted, the homogeneity strength condition of EC8 is satisfied, i.e. the ratio between the maximum and minimum overstrength of the braces $\Omega_{s,max}/\Omega_{s,min}$ is never greater than 1.25.

After the cross-section size of the braces has been assigned, the design internal forces of beams and columns of the braced frame are determined by the capacity design criteria described in EC8 [22, 45]. In general, the design axial force $N_{Ed,i}$ of beams and columns at the i-th storey is calculated by the following equation

$$N_{Ed,\,i} = N_{Ed,\,G,i} + 1.1\,\gamma_{ov}\,\Omega_{s,min}\,N_{Ed,E,\,i} \tag{18}$$

where $N_{Ed,G,i}$ and $N_{Ed,E,i}$ are the axial forces provided by gravity loads and seismic forces, and the coefficient γ_{ov} is the steel overstrength factor assumed equal to 1.2 as suggested in [9]. Beams sustain only the axial force $1.1\,\gamma_{ov}\,\Omega_{s,min}\,N_{Ed,E,i}$ ($N_{Ed,G,i} = 0$) and the bending moment caused by the gravity loads. For columns, $N_{Ed,G,i}$ is calculated according to the tributary area concept and $N_{Ed,E,i}$ is given by the design seismic analysis. Bending moments on columns have been neglected. The cross-section size of the beams is determined by two design conditions: the bending moment must be smaller than the plastic moment resistance reduced because of the axial force and the axial force must be smaller than the buckling resistance reduced because of the bending moment. The buckling resistance of the beams is calculated assuming that the buckling about the weak axis is restrained by the deck. The cross-section size of the columns is obtained by equating their axial force to their buckling resistance. Both moment and buckling resistances are determined according to the strength criteria stipulated in Eurocode 3 (EC3) assuming the partial safety coefficients γ_{M0} and γ_{M1} are equal to unity [46].

The beam-to-column connections of the columns that do not belong to the braced frame are pinned. These columns sustain only the axial force provided by gravity loads (*gravity columns*). The design axial force is evaluated according to the tributary area concept and considering the load per square meter of the non-seismic situation equal to 9.2 kN/m^2. The cross-section size of these columns is obtained by equating their axial force to their buckling resistance evaluated according to EC3 assuming γ_{M1} = 1.0.

Dual Systems

According to EC8, the lateral forces applied on the dual system are distributed between the CBFs and MRFs according to their lateral stiffness. Further, EC8 recommends that the MRFs and the CBFs of the dual system are designed in accordance with the provisions stipulated for the two structural types independently considered. Thus, the design of the braced frames is performed in compliance with the previous section.

For moment resisting frames, the dissipative zones are located at the ends of all the beams, at the base of the first storey columns and at the upper end of the top storey columns [47-51]. The minimum required value of the beam plastic modulus is obtained by equating the design bending moment M_{Ed} to the plastic bending moment $M_{\mathrm{pl,Rd}}$. Design bending moments are calculated as the sum of the effects of the gravity loads and seismic actions considered in the seismic design situation. As recommended in EC8, beam sections are selected so that the design axial force and the shear force do not decrease the full plastic moment and the rotation capacity at the plastic hinge. The beams designed are also verified to sustain the gravity loads of the non-seismic design situation and limit the deflection to the reference value reported in EC3 for the serviceability limit state. The overstrength factors Ω^{MRF} of the dissipative zones of the MRFs are calculated as the ratio of the full plastic resistance in bending to the design bending moment. The design internal forces of the non-dissipative zones of columns are obtained, according to the capacity design criteria, by the following relations

$$N_{\mathrm{Ed,\,i}} = N_{\mathrm{Ed,\,G,i}} + 1.1\,\gamma_{\mathrm{ov}}\,\Omega^{\mathrm{MRF}}_{\mathrm{min}}\,N_{\mathrm{Ed,E,\,i}} \tag{19a}$$

$$M_{\mathrm{Ed,\,i}} = M_{\mathrm{Ed,\,G,i}} + 1.1\,\gamma_{\mathrm{ov}}\,\Omega^{\mathrm{MRF}}_{\mathrm{min}}\,M_{\mathrm{Ed,E,\,i}} \tag{19b}$$

$$V_{\mathrm{Ed,\,i}} = V_{\mathrm{Ed,\,G,i}} + 1.1\,\gamma_{\mathrm{ov}}\,\Omega^{\mathrm{MRF}}_{\mathrm{min}}\,V_{\mathrm{Ed,E,\,i}} \tag{19c}$$

Then, the column cross-sections are selected so that two conditions are verified: (i) the design bending moment is lower than the flexural strength reduced because of the axial force; (ii) the design axial force is lower than the buckling resistance reduced because of the design bending moment. Both flexural and buckling resistances are calculated according to EC3 assuming $\gamma_{M0} = \gamma_{M1}$ = 1.0 [46].

NUMERICAL ANALYSES

The seismic behaviour of the considered structures is evaluated by incremental nonlinear dynamic analysis. The single nonlinear dynamic analysis is carried out by means of the OpenSEES program [52]. The peak ground acceleration a_{g} is scaled in step of 0.04g in order to estimate the peak ground acceleration corresponding to the attainment of the ultimate ductility in braces.

Modelling of Structures

The numerical analyses are carried out on two-dimensional models rather than on three-dimensional models (as an example see Fig. **4** for dual systems). This simplification is possible because of the symmetry of the structure [53-56] and because no in-plan variation of the nominal mass, stiffness and strength distribution is considered [57-59]. The symmetry in the dynamic and mechanical characteristics of the system allows the adoption of a model which represents half the structure of the building.

The braces are modelled by four "nonlinearBeamColumn" elements. The cross-section is divided into 20 fibres. The hysteretic behaviour of steel is represented by the model of Menegotto-Pinto [60, 61] with kinematic and isotropic hardening. An initial camber displacement of 0.1% of the brace length is applied at brace mid-length [62-64]. The corotational theory is used to represent the moderate to large deformation effects of inelastic buckling of the brace [65]. Beams and columns of the CBFs are expected to remain elastic and, therefore, are modelled by means of "elasticBeamColumn" elements. When dual systems are considered, the plastic behaviour of beams and columns of the moment resisting frames is modelled by means of "beamWithHinges" elements. The depth of the plastic hinge is assumed to be equal to the height of the cross-section of the considered member. The viscous damping forces are obtained through the formulation proposed by Rayleigh. In particular, the stiffness coefficient is applied to the initial stiffness matrix of the elements. A viscous damping ratio equal to 0.05 is fixed for periods equal to those of the first and third modes of vibration of the 8- and 12-storey structures, and for periods equal to those of the first and second modes of vibration of the 4-storey structures. For each structure the analysis is performed considering the P-Δ effects.

Seismic Input

The seismic input is constituted by ten artificial accelerograms. The artificial accelerograms are generated by means of the software SIMQKE [66]. The single artificial accelerogram is defined by a stationary random process modulated by means of a compound intensity function [67]. The total duration of each accelerogram is fixed equal to 30.5 s. The earthquake rise time is 4 s; the parameter IPOW of the first branch of the intensity function and the parameter ALFA0 of the third one are assumed equal to 2 and 0.25, respectively (Fig. **5a**). The duration of the stationary part of the accelerograms is equal to 7.0 s and, therefore, is lower than the minimum value suggested by EC8, i.e. 10 s. The

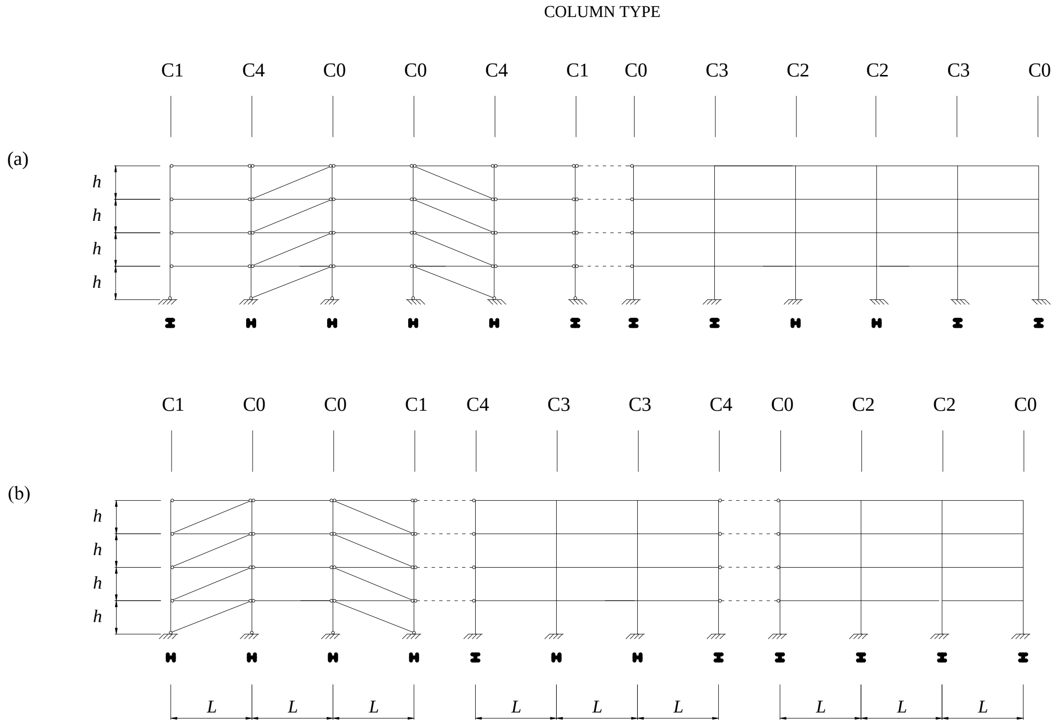


Fig. (4). Numerical models for seismic action acting along (**a**) *x*-direction, (**b**) *y*-direction.

adopted value has resulted from a previous investigation in which natural and artificial accelerograms were compared in terms of input energy spectra, Arias intensity, frequency content and number of equivalent cycles [68]. The comparison between the elastic response spectrum of each accelerogram and the elastic spectrum proposed by EC8 for soil C is shown in (Fig. **5b**).

Response Parameter

For each nonlinear dynamic analysis, the response of the frame is expressed in terms of the required ductility of the braces. To be consistent with the approach followed by Tremblay in deriving the ultimate ductility μ_f, the brace ductility demand μ_d is calculated here as the sum of the maximum absolute values of the shortening δ^- and elongation δ^+ divided by δ_y. Then, the damage index is calculated by Equation (1). The peak ground acceleration is increased in step of 0.04g until the maximum damage index along the height of the building is equal to 1.

VALIDATION OF THE PREDICTION OF THE DAMAGE DISTRIBUTION

The damage distribution is first predicted with reference to the single accelerogram. Consequently, in order to evaluate the overstrength factor, the storey shear V_{Ed} of Equation (7) is calculated by modal response spectrum analysis considering the elastic response spectrum corresponding to the accelerogram under examination. The same spectrum is adopted to evaluate the required interstorey displacements of Equation (8). The expected damage index DI^e is then calculated with reference to each accelerogram by Equation (2). Finally, the mean values (with respect to the 10 accelerograms) of the expected damage index are calculated. These values are scaled so that the maximum damage index is equal to 1 and are compared to the actual values DI^{dyn} obtained by nonlinear dynamic analyses.

Overstrength Factor of the Considered Frames

Fig. (**6**) shows the heightwise distribution of the normalised overstrength factor ($\Omega_s/\Omega_{s,min}$) for some simple and dual diagonal braced frames designed according to EC8. Specifically, at each storey the maximum, minimum and mean values of the overstrength factors obtained with reference to the considered accelerograms are reported. In 12-storey simple concentrically braced frames designed by the lateral force method of analysis (12EC8s), the heightwise distribution of the overstrength factor is quite scattered. The overstrength is generally low at the upper storey because the lateral force method of analysis significantly underestimates the design seismic actions at the higher storeys of the building.

The heightwise distribution of the overstrength factor is more uniform in 12-storey simple concentrically braced frames designed by modal response spectrum analysis. However, in these buildings, the homogeneity strength condition is not rigorously satisfied. This aspect is mainly due to the difference between the design spectrum (where a non-zero minimum value of the pseudo-acceleration is usually considered) and the response spectrum corresponding to the accelerogram under examination.

When dual systems are considered, the seismic forces at the upper storeys are mainly sustained by the moment resisting frames and the cross-section of the braces are generally oversized in order to satisfy the upper limit on the normalised slenderness considered in EC8. For this reason in 12-storey dual systems the overstrength factor is generally higher at the upper storey.

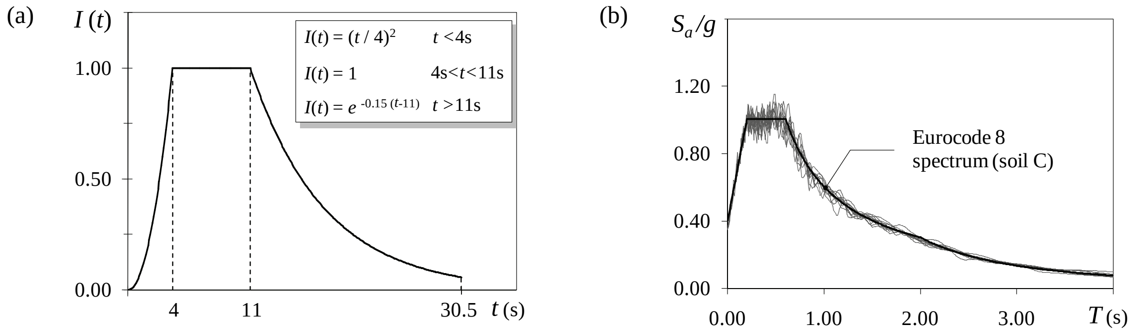


Fig. (5). Artificial accelerograms: (**a**) Compound intensity function; (**b**) comparison between elastic response spectra of the accelerograms and EC8 elastic spectrum.

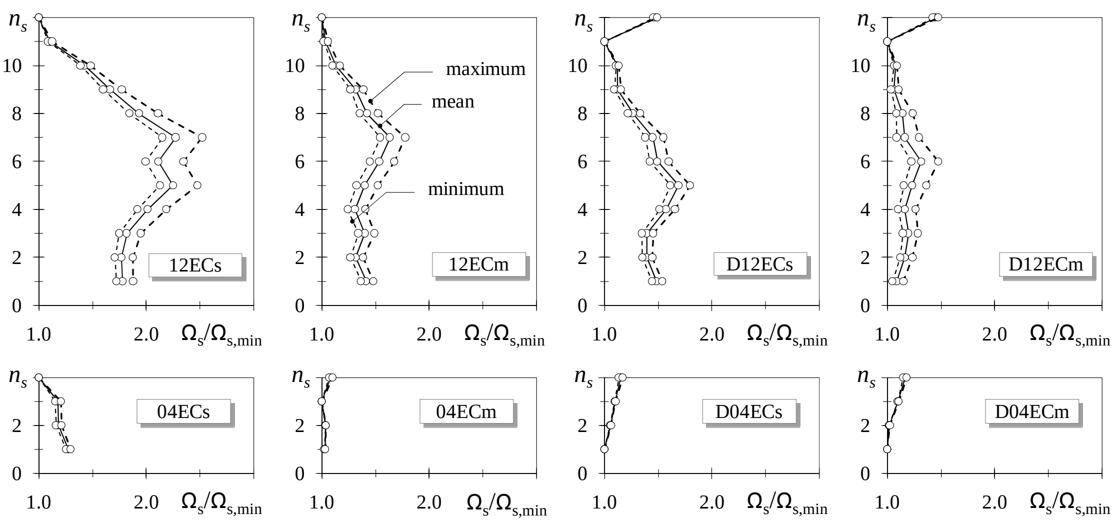


Fig. (6). Heightwise distribution of the overstrength factor.

In 4-storey systems the homogeneity strength condition is always satisfied, regardless of the adopted method of analysis.

Damage Distribution Capacity Factor of the Considered Frames

Fig. (**7**) shows the heightwise distribution of the damage distribution capacity factor for the systems described in the previous section. The evaluation of the *DDC* factor is almost independent of the considered accelerograms. In fact, the minimum, maximum and mean values of the *DDC* factor at each storey are almost coincident. For simple steel frames with diagonal braces the minimum value of the *DDC* factor along the height of the frame is quite low (close to 0.05). Further, in 12-storey frames this minimum value is reached at the upper storey, i.e. where the premature yielding of the brace in tension occurs. Thus, high values of the damage index are expected at the upper storey of these systems.

Instead, when dual systems are considered, the *DDC* factor is quite uniform and the value DDC_{min} is larger than 0.20 and 0.30 in 4- and 12- storey systems, respectively. Note that a value of DDC_{min} equal to 0.3 was suggested in a previous study for the design eccentrically braced dual systems [26-27] in order to ensure a collapse mechanism characterised by high values of the damage index along the height of the building.

Effectiveness of the Proposed Formulation

Fig. (**8**) shows the comparison between the damage index evaluated by means of the nonlinear dynamic analysis (black dots), assumed as the actual value, and that provided by the proposed method (white dots). The comparison is restricted to some of the analysed 4- and 12-storey frames. The proposed expressions are able to predict accurately the response of systems in which the damage is mainly restricted to a few storeys. In the other cases, the method is still able to predict an almost uniform distribution of the damage, although some not negligible scattering between actual and expected values can be found at some storeys.

The comparison between expected and actual values of the damage index is represented for all the considered frames in (Fig. **9a**). The *x*- and *y*-coordinates of the dots represented in this figure are the actual and predicted values of the damage index DI at each storey. Dots lying along the bisector denote that the proposed method predicts a value of the damage index equal to that obtained by the nonlinear dynamic

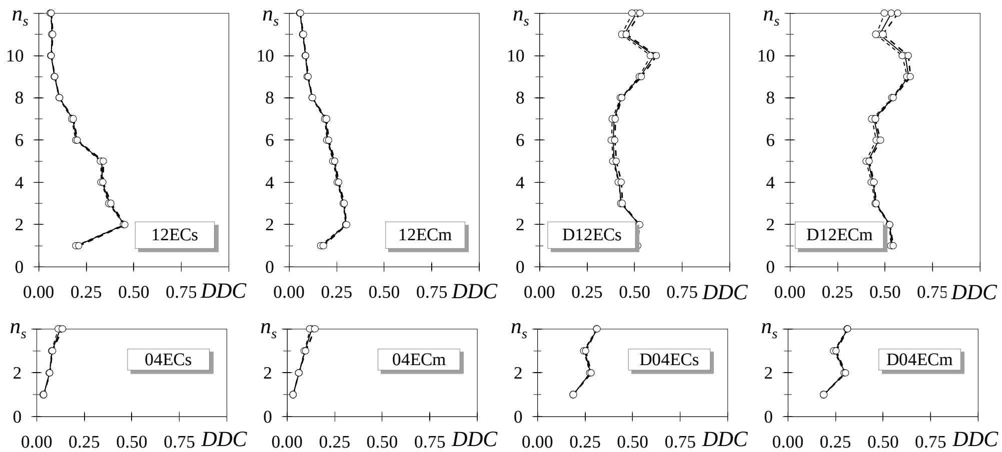


Fig. (7). Heightwise of the damage distribution capacity factor.

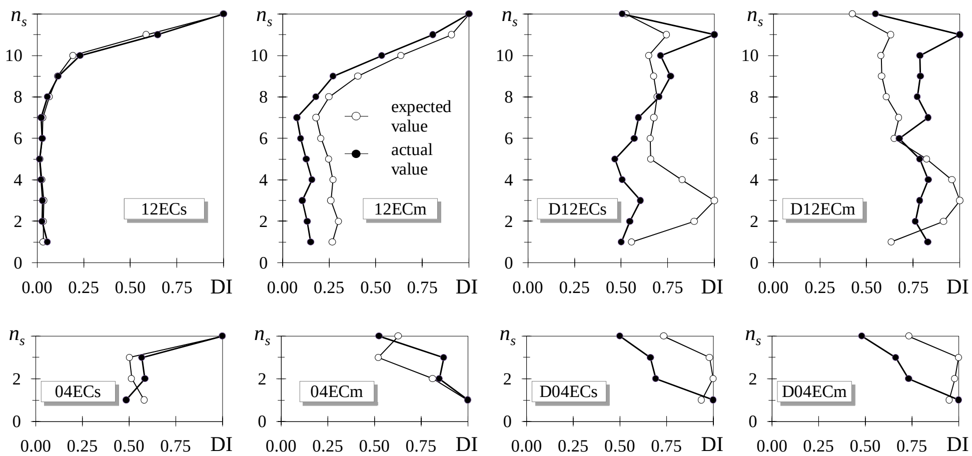


Fig. (8). Comparison between the actual and expected value of the damage index.

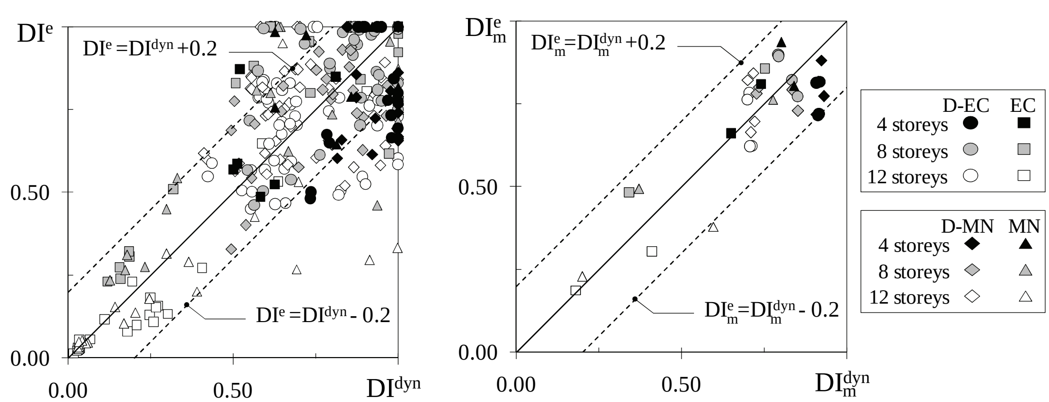


Fig. (9). Comparison between predicted and actual values of: (**a**) the damage index of braces of single storey (**b**) the average damage index.

analysis. This figure shows that the absolute value of the difference between actual and expected damage indexes is generally lower than 0.20. Better results are obtained in terms of the mean value of the damage index (DI_m) of all the braces of the analysed frame (Fig. **9b)**.

CONCLUSION

In this paper a relation previously proposed to predict the heightwise damage distribution of eccentrically braced frames at collapse is extended to frames with concentric diagonal bracings.

The damage distribution is calculated as a function of the brace overstrength factor and the damage distribution capacity factor.

The proposed expression is validated with reference to 12 simple braced frames and 12 dual braced systems designed according to different methods and characterised by a different number of storeys.

The numerical investigation shows that the proposed expression is able to predict accurately the nonlinear dynamic response of systems in which the damage is mainly restricted to a few storeys. If this is not the case, the method can provide noticeable results although some not negligible scattering between actual and expected values can be found at some storeys.

CONFLICT OF INTEREST

The author(s) confirm that this article content has no conflicts of interest.

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