Proposal and validation of a design procedure for concentrically braced frames in the chevron configuration

Francesca Barbagallo | Melina Bosco | Edoardo M. Marino | Pier Paolo Rossi

Department of Civil Engineering and Architecture, University of Catania, Catania, Italy

Correspondence
Melina Bosco, Department of Civil and Environmental Engineering, University of Catania, Via Santa Sofia, 64, 95125 Catania, Italy.
Email: mbosco@dica.unict.it

Abstract
In the recent past, several research studies have highlighted that the prescriptions reported in codes for the design of structures with concentric braces in the chevron configuration are often not effective in preventing yielding or buckling of non-dissipative members and in ensuring collapse mechanisms characterised by uniform damage of braces. To investigate the reasons of these deficiencies, in this paper the seismic response of concentrically braced structures designed according to procedures reported in the literature and in the European seismic code is first examined. Then, a new design procedure is proposed, in which the innovative aspects are mainly related to the estimation of the bending moments in columns and to the formulation of requirements on the stiffness of braced beams and columns. The impact of the proposed procedure on the structural costs is computed on a large number of buildings characterised by different occupancy types and geometric properties. The seismic performance of these structures is evaluated by incremental nonlinear dynamic analysis and discussed at the achievement of the significant damage and near-collapse limit states.

Keywords
bending moments in columns, concentrically braced frames, design procedures

1 | INTRODUCTION

Design provisions and philosophies of concentrically braced frames (CBFs) vary substantially around the world. In regard to the design philosophy commonly adopted in Europe, which is intended to promote an elastic behaviour of braced beams and columns, several research studies have highlighted some critical issues related to the design of the members of this structural type. In some of these research studies, attention has been focused on the importance of the design prescriptions requiring an almost uniform distribution of the overstrength factor of braces in order to promote a collapse mechanism characterised by a pronounced inelastic behaviour of braces of all storeys. Often, these studies differ because of the method used to evaluate the overstrength factor of braces.1–3 According to Eurocode 8 (EC8),1 the overstrength factor of braces is defined as the ratio of the design resistance of the brace (indicated by symbol \(N_{pl,Rd}\)) to the design value of the axial force in the seismic design situation. In the case of single diagonal braces, the design resistance of the brace is always intended as the plastic resistance of the brace cross-section, whereas the resistance of the brace in compression is neglected. In

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the case of braces in the chevron configuration, instead, the symbol associated with the design resistance of the brace \( N_{pl,Rd} \) has raised some doubts regarding the expected calculation of this parameter. According to some researchers, for example,\(^2\) the design resistance of the brace should be determined as the plastic resistance of the cross-section (as in structures with diagonal braces), whereas according to others, for example,\(^3\) the design resistance of the brace in the chevron configuration should be calculated as the buckling resistance of the member. In the attempt to propose a more effective predictor of a collapse mechanism characterised by a uniform damage of braces, some other researchers\(^4\) have also suggested the calculation of the overstrength factor on the basis of the lateral storey resistance and design lateral storey shear force of a couple of braces in the chevron configuration. Specifically, the lateral storey resistance of the brace in tension should correspond to the plastic resistance of the cross-section, whereas the lateral storey resistance of the brace in compression should correspond to the postbuckling resistance of the member.

Still with reference to the design prescriptions regarding the overstrength factor, some researchers have suggested that the uniformity of the overstrength factor should be verified ignoring the top storey braces.\(^5,6\) Indeed, in practical applications these latter braces are generally oversized to fulfill the design code prescriptions regarding the upper limit of the normalised slenderness of braces. Therefore, if the uniformity of the overstrength factor were verified with respect to all braces of the structure, the overstrength of the top storey braces would lead to an increase of the cross-section of the braces of all the other storeys and thus to an increase in the costs of the structure. In addition, Costanzo et al.\(^5\) state that ignoring the top storey braces in the verification of the uniformity of the overstrength factors would also promote a shear-type lateral displacement profile and would avoid damage concentration at a few storeys.

In regard to the inelastic behaviour of braces in tension, some research studies have highlighted the importance of the flexural stiffness of the braced beams.\(^2,7,8\) Indeed, flexible braced beams limit the inelastic elongation of braces under tension and sometimes prevent these members from yielding. Conversely, stiff beams ensure large plastic elongations and thus high energy dissipation prior to brace fracture.

In regard to the design of beams in terms of strength, seismic codes require that beams sustain axial forces, shear forces and bending moments calculated under the hypothesis that the brace in tension has yielded and that the brace in compression is in its postbuckling range of behaviour. Prescriptions given in codes, however, differ because of the relations proposed to estimate the ultimate resistance of the brace in compression and because of the requirements that beam cross-section has to satisfy. According to AISC 341,\(^9\) beams should be braced to satisfy the requirements for moderately ductile members. In EC8, instead, no specific prescription is given in this regard, and beams are considered as fragile members. Based on the results of previous experimental tests and numerical studies indicating that a limited yielding beam is not detrimental to nonlinear response,\(^10\) a different approach with yielding beams is suggested by Tan et al.\(^11\)

Several research studies have also proved that the rules for the application of the capacity design principles are often not effective in preventing yielding or buckling of the columns of the braced frame.\(^7,12-16\) In this regard, Longo et al.\(^12\) suggest that the design axial forces in columns should be evaluated assuming that braces have yielded at all storeys, whereas Bosco et al.\(^15\) and Kumar et al.\(^16\) point out that the design procedures reported in seismic codes or in literature generally neglect or significantly underestimate the bending moments of columns. Indeed, these bending moments are negligible when braces are elastic, but they can be significant when braces are in their inelastic range of behaviour and damage concentrates at a few storeys. The high values of these bending moments often undermine the seismic performance of CBFs because yielding or buckling of columns occurs prior to the full exploitation of the deformation capacity of braces. To estimate the design bending moments in columns, some researchers\(^15,16\) suggest to perform additional structural analyses on numerical models in which braces are removed from single storeys; instead, some other researchers\(^6\) and some codes suggest that the design bending moments of the columns should be evaluated as 20% of their flexural plastic resistance.

In this paper, the authors investigate the above issues regarding the design of concentrically braced structures with braces in the chevron configuration and pinned beam-to-column connections, as is usual in Europe. In particular, to shed some light on the correlation between the selected brace overstrength measure and the mean damage of braces, CBFs are first designed according to design procedures available in the literature or in seismic codes and embedding different definitions of the brace overstrength factor. The seismic response of these systems is evaluated by incremental nonlinear dynamic analysis and the attention is focused on the response corresponding to the achievement of the significant damage (SD) and near collapse (NC) limit states in the structure. The results of these numerical analyses are also examined to assess the effectiveness of the rules for the application of the capacity design principles to beams and columns, the rules being applied as reported in the considered design procedures. Then, based on the observations resulting from the above numerical analyses, a new design procedure is proposed. The innovative aspects of the proposed procedure are mainly related to the estimation of the bending moments in columns and to the formulation of a stiffness requirement for the braced beams and columns. This latter requirement limits the reduction of the lateral storey stiffness after buckling of braces and promotes a pronounced inelastic behaviour of braces in tension and compression at all storeys of the building.
The proposed design procedure is applied to four groups of buildings characterised by different occupancy types and geometric properties. The seismic performance of these structures is evaluated by incremental nonlinear dynamic analysis and discussed at the achievement of the SD and NC limit states. The impact of the proposed procedure on the structural costs of the braced buildings is also evaluated.

2 DESIGN OF CHEVRON BRACED FRAMES ACCORDING TO EXISTING PROCEDURE

Different design procedures have been proposed in the literature or in seismic codes to design CBFs in the chevron configuration. In this research study, three design procedures are considered and briefly recalled in the following subsections.

2.1 Eurocode 8 design procedure

According to EC8, CBFs in the chevron configuration are designed by means of a force-based design procedure. Internal forces of members are determined by a modal response spectrum analysis assuming a behaviour factor equal to 2.5. The minimum cross-sectional area of braces is determined by equating the design buckling resistance of braces $N_{Rd,br}$ to the internal axial forces $N_{Ed,br}$ determined by the design structural analysis. These latter internal forces are the sum of the internal forces due to the seismic forces $N_{Ed,E}$ and those produced by the gravity loads in the seismic design situation $N_{Ed,G}$, that is,

$$N_{Rd,br} \geq N_{Ed,br} = N_{Ed,G} + N_{Ed,E}. \quad (1)$$

The brace overstrength $\Omega$ is calculated at each storey as the ratio of the buckling resistance of braces to the internal axial forces of braces, that is,

$$\Omega = \frac{N_{Rd,br}}{N_{Ed,br}} \quad (2)$$

To promote a dissipative behaviour of the structure, the ratio of the overstrength factor of the single $i$-th storey to the minimum overstrength ($\Omega_{\text{min}}$) should be lower than 1.25, that is,

$$\frac{\Omega_i}{\Omega_{\text{min}}} \leq 1.25 \quad i = 1, \ldots, n. \quad (3)$$

Design internal forces in beams and columns are determined according to simplified rules of application of the capacity design principles. Specifically, axial forces are calculated as the sum of the contribution due to the gravity loads in the seismic design situation and the contribution due to the seismic actions amplified to take account of the minimum overstrength of braces and overstrength of steel $\gamma_{ov}$. Bending moments in columns are obtained by means of the elastic analysis of the frame. The column cross-section and steel grade are chosen so as to resist the combined effects of axial forces and bending moments, that is,

$$N_{Rd,c} \left( M_{Ed,c} \right) \geq N_{Ed,G} + 1.1 \gamma_{ov} \Omega_{\text{min}} N_{Ed,E}, \quad (4)$$

where $N_{Rd,c}$ is the design buckling resistance of the column reduced to take into account the design bending moment $M_{Ed,c}$ of the column and calculated as reported in Eurocode 3.

The beam is designed to sustain the gravity loads of the nonseismic design situation without the contribution of the braces. In addition, the beam is designed to resist the shear force $V_{Ed,b}$ and the bending moment $M_{Ed,b}$ produced by the unbalanced vertical force transferred by the tension and compression braces and calculated assuming that the brace in tension has yielded ($N_{pl,br}$) and that the brace in compression is in the postbuckling range of behaviour ($N_{u,br} = 0.3 N_{pl,br}$), that is,

$$V_{Ed,b} = (N_{pl,br} - N_{u,br}) \sin \alpha, \quad M_{Ed,b} = (N_{pl,br} - N_{u,br}) \sin \alpha \frac{L}{4}. \quad (5)$$
where $L$ is the length of the braced span and $\alpha$ is the slope of the brace to the horizontal axis. As stipulated in Eurocode 3,\textsuperscript{25} if the shear force does not exceed 50% of the design plastic shear resistance $V_{pl,Rd}$, no reduction of the resistances defined for bending and axial force should be considered. If $V_{Ed,b}$ exceeds 50% of $V_{pl,Rd}$, the design resistance of the cross-section to combinations of bending moment and axial force (to be compared to $M_{Ed,b}$) is calculated using a reduced yield strength for the shear area.

### 2.2 Marino and Nakashima design procedure

The procedure proposed by Marino and Nakashima (MN design procedure) mainly differs from that of EC8 because of the adopted value of the behaviour factor (3.5 instead of 2.5) and because the design of braces and the definition of the storey overstrength factor is proposed with reference to the ultimate response of braces.\textsuperscript{3} Owing to this, the design of braces is carried out by equating the design storey shear ($V_{Ed}$) to the storey shear strength corresponding to the plastic resistance of the brace in tension ($N_{pl,br}$) and to the postbuckling resistance of the brace in compression ($N_{u,br}$), that is,

$$ (N_{pl,br} + N_{u,br}) \cos \alpha \geq V_{Ed}. \quad (6) $$

Further, differently from EC8, the postbuckling resistance of the brace in compression is calculated analytically as a function of the normalised slenderness of the braces.

The storey overstrength factor is calculated as the ratio of the lateral storey strength to the design storey shear, that is,

$$ \Omega = \frac{(N_{pl,br} + N_{u,br}) \cos \alpha}{V_{Ed}}, \quad (7) $$

and the ratio of the overstrength factor of the single storey to the minimum overstrength factor is forced to be lower than 1.25.

The axial forces in columns and in beams are calculated by equilibrium conditions considering two different situations. In the first situation, the axial forces of braces in tension and in compression are assumed to be equal to the buckling resistance of the braces ($N_{b,br}$); in the second situation, the brace in tension is assumed to be yielded and fully hardened ($1.1 N_{pl,br}$), whereas the brace in compression is in the postbuckling range of behaviour. These equilibrium equations lead to the following estimate of the axial forces in columns ($N_{Ed,c}$) and beams ($N_{Ed,b}$):

$$ N_{Ed,c,i} = N_{Ed,G} + \gamma_{ov} \max \left\{ \sum_{j=i}^{n} \frac{1.1 N_{pl,br}^{(j)} - N_{u,br}^{(j)}}{2} \sin \alpha + \sum_{j=i+1}^{n} N_{u,br}^{(j)} \sin \alpha ; \sum_{j=i+1}^{n} N_{b,br}^{(j)} \sin \alpha \right\}, \quad (8) $$

$$ N_{Ed,b,i} = \gamma_{ov} \frac{1.1 (N_{pl,br,i} + N_{pl,br,i+1}) + N_{u,br,i} - N_{u,br,i+1}}{2} \cos \alpha. \quad (9) $$

In keeping with the prediction of the axial forces, the shear force and bending moments caused in beams by the unbalanced vertical force transferred by braces are calculated assuming that the brace in tension has yielded and is fully hardened and that the brace in compression is in its postbuckling range of behaviour, that is,

$$ V_{Ed,b} = \gamma_{ov} (1.1 N_{pl,br} - N_{u,br}) \sin \alpha, \quad M_{Ed,b} = \gamma_{ov} (1.1 N_{pl,br} - N_{u,br}) \sin \alpha \frac{L}{4}. \quad (10) $$

### 2.3 Costanzo, D’Aniello and Landolfo design procedure

The procedure proposed by Costanzo, D’Aniello, and Landolfo (CDL design procedure) suggests that the design structural analysis should be carried out with a behaviour factor equal to 4. With the sole exception of the top storey, the cross-section of braces is selected to fulfil Equation (1). The braces of the top storey, instead, are designed to remain elastic in the seismic design situation so that this storey behaves like an outrigger system and promotes a shear-type displacement
profile. Owing to this, the buckling resistance of the top storey braces shall be higher than the axial force determined by the design structural analysis multiplied by the behaviour factor. To promote a dissipative behaviour of the structure, the brace overstrength $\Omega$ is calculated at each storey (with the exception of the top storey) as the ratio of the buckling resistance of braces to the internal axial forces of braces, and the ratio of the overstrength factor of the single storey to the minimum overstrength factor is forced to be lower than 1.25.

Non-dissipative members are designed to resist the following two combinations of internal forces. The axial forces of the first combination are calculated as reported in Equation (4), as the sum of the axial forces due to the gravity loads of the seismic design situation and the axial forces due to the seismic actions amplified to take account of the minimum overstrength of braces and of the overstrength of steel ($\gamma_{ov}$). However, in this combination the brace overstrength is calculated based on the plastic resistance of braces, and the minimum overstrength factor is

$$\Omega = \Omega_{pl} = \min \left( \frac{N_{pl,br,i}}{N_{Ed,br,i}} \right) \quad i = 1, \ldots, n - 1. \quad (11)$$

The internal forces of the second combination are calculated by means of equilibrium conditions, assuming that the braces in tension have yielded whereas those in compression are in their postbuckling range of behaviour, that is,

$$N_{Ed,c,i} = N_{Ed,G} + \sum_{j=i}^{n-1} \frac{\gamma_{ov} N_{pl,br}^{(j)} - N_{u,br}^{(j)}}{2} \sin \alpha + \sum_{j=i+1}^{n-1} N_{u,br}^{(j)} \sin \alpha. \quad (12)$$

In this regard, Costanzo et al. note that the contribution due to the axial forces in the braces of the top storey should be neglected, and the postbuckling resistance of the braces at the other storeys should be calculated as

$$N_{u,br} = \gamma_{ov} 0.3 N_{b,br}. \quad (13)$$

Further, in the case of the second combination, the resistance and the stability of columns should be verified considering the simultaneous presence of a bending moment equal to 20% of the plastic bending resistance of the column cross-section, in keeping with recommendations of other seismic codes.17

Beam cross-sections should be designed based on both strength and stiffness requirements. The strength requirement is similar to that reported in EC8, with the sole exception that the postbuckling resistance is calculated according to Equation (13). The stiffness requirement, instead, prescribes that the beam-to-brace stiffness ratio $K_F$ should be larger than 0.2, that is,

$$K_F = \frac{k^*_b}{k_{br}} \geq 0.20, \quad (14)$$

where $k_{br}$ is the vertical stiffness of the bracing system and $k^*_b$ is the flexural stiffness of the beam accounting for the deformability of supports. More details regarding this design procedure can be found in.5

### 3 | CASE STUDIES

To investigate the effectiveness of the above design procedures, four groups of buildings characterised by square-plan layout have been selected as case studies. For the sake of simplicity, geometric and mass properties are equal at all storeys. All the buildings are founded on soft soil (soil type “C” in EC8). The structural scheme is defined by the intersection of two sets of four three-bay frames located along two orthogonal directions (Figure 1). Concentric braces are located in the central span of the external frames and arranged in the chevron-braced configuration. The length $L$ of each bay is equal to 8.0 m, and the number of storeys is equal to either, 4, 8 or 12. The main characteristics of the buildings belonging to the four considered groups are summarised in Table 1. Specifically, the occupancy type, interstorey height $h$, characteristic values of vertical dead ($g_k$) and live ($q_k$) loads and vertical loads in the seismic design situation ($g_k + \psi_2 q_k$) are reported. Independently of the considered group of buildings, HSS cross-sections are used for braces and wide flange cross-sections are used for beams and columns. As common in practical applications, the same column cross-section is used for columns belonging to two adjacent storeys.
The seismic design load is calculated as a function of a peak ground acceleration (PGA) equal to 0.35g and entirely resisted by braces. Columns belonging to the braced frames (columns C0 in Figure 1) are designed to sustain the axial forces transferred by braces, whereas gravity columns (columns C1 and C2 in Figure 1) are designed to sustain the gravity loads of the nonseismic design situation. When the CDL design procedure is adopted, a design bending moment equal to 20% of the cross-section plastic resistance is assumed for columns C0, C1 and C2.

The designed structures are identified by a label referring to the design procedure (‘EC8’, ‘MN’ or ‘CDL’), the number of storeys (‘04’, ‘08’ or ‘12’) and the group of buildings (‘#1’, ‘#2’, ‘#3’ or ‘#4’). The adopted cross-sections and the design internal forces of members of all the designed structures are available at the website http://www.dica.unict.it/users/mbosco/VCBF-CapacityDesign.htm

### 4 DETAILS ON APPLICATION OF THE DESIGN PROCEDURES

To match the design buckling resistance of the brace (with HSS cross-section) by means of the adopted numerical model, analytical expressions of the design buckling resistance of the brace have been derived as a function of the plastic resistance and normalised slenderness of the brace. Specifically, the ratio of the buckling resistance to the plastic resistance ($\chi_b$) of the brace (shown in Figure 2A) is calculated as a function of the normalised slenderness of the brace $\bar{\lambda}$ by means of the

![Figure 2](image-url)
\[
\chi_b = 0.0308 \bar{\lambda}^6 - 0.2939 \bar{\lambda}^5 + 0.9952 \bar{\lambda}^4 - 1.301 \bar{\lambda}^3 + 0.2767 \bar{\lambda}^2 + 0.0447 \bar{\lambda} + 0.9561. \tag{15}
\]

Similarly, when the MN design procedure is used, the ratio of the postbuckling resistance to the plastic resistance \(\chi_u\) of the brace is calculated by means of the following relations for steel grades S235 and S355:

\[
\chi_u = 0.0138 \bar{\lambda}^6 - 0.189 \bar{\lambda}^5 + 1.05 \bar{\lambda}^4 - 3.04 \bar{\lambda}^3 + 4.83 \bar{\lambda}^2 - 4.04 \bar{\lambda} + 1.48 f_y = 235\text{MPa}, \tag{16}
\]

\[
\chi_u = 0.0129 \bar{\lambda}^6 - 0.180 \bar{\lambda}^5 + 1.02 \bar{\lambda}^4 - 2.99 \bar{\lambda}^3 + 4.85 \bar{\lambda}^2 - 4.19 \bar{\lambda} + 1.62 f_y = 355\text{MPa}. \tag{17}
\]

The procedure used to calibrate the analytical expression is formally equal to that described in other studies. In particular, the procedure given in is used with the sole exception that the ductility capacity is calculated according to the formulation proposed by Marino. Figure 2B shows a comparison between the values of \(\chi_u\) obtained by means of the proposed relations, the values corresponding to the postbuckling resistance by Equation (13), that is, \(\chi_u = 0.3 \chi_b\), and the values prescribed in EC8 \(\chi_u = 0.3\). To show the accuracy of the proposed analytical relations, the values of \(\chi_b\) and \(\chi_u\) obtained by numerical simulation of the nonlinear response of a large set of braces with HSS cross-section and yield strength \(f_y\) equal to either 235 or 355 MPa are reported by grey and white dots in the same figures. The figure shows that, in the case of braces characterised by normalised slenderness higher than 0.75, the expression prescribed in EC8 provides the highest values of \(\chi_u\) and thus the lowest unbalanced vertical forces on beams. The highest unbalanced vertical forces are given by the relations in the MN design procedure, while intermediate unbalanced vertical forces are obtained by Equation (13). An opposite trend is highlighted for braces with normalised slenderness lower than 0.65. This latter observation is, however, of minor importance for the present paper because only a few braces among those used in this research are characterised by normalised slenderness lower than 0.65. Finally, to be consistent with the model of the yield strength of steel assumed in the numerical analyses (where no scattering is considered for this parameter), the overstrength of steel \(\gamma_{ov}\) is fixed equal to 1.0.

5 | NUMERICAL ANALYSES

The seismic response of the considered structures has been determined by means of incremental nonlinear dynamic analysis. The single numerical analysis is carried out by the OpenSees computer program.

5.1 | Seismic input and numerical modelling

The seismic input consists of 10 artificially generated accelerograms that are compatible with the elastic response spectrum defined in EC8 for soil type C, PGA equal to 0.35 g and equivalent viscous damping ratio equal to 5%. The seismic input has been generated by the SIMQKE computer program. Each ground motion is 30.5 s long and enveloped by a three-branch compound function with a strong motion phase of 7 s.

Owing to the symmetry of the structure, a planar model representing half of the structure is considered in the numerical analysis (Figure 1B). Braces are modelled by means of four nonlinear beam–column elements, as proposed by Uriz et al. To simulate the effects of imperfections, the initial camber is set equal to 0.1% times the length of the brace and the nodes of the four elements of braces are arranged on a sinusoidal curve. The corotational formulation is applied to consider the geometric nonlinearities. The brace cross-section is discretised into fibres, namely five fibres over the thickness of each flange and five fibres over the depth of the web. Four fibres are also added in the corners between flanges and web to match perfectly the mass properties of the cross-section. The Giusfrè-Menegotto–Pinto uniaxial material model (Steel02 in OpenSees) is used to simulate the response of steel. Beams and columns are considered as fragile members in keeping with EC8 design philosophy and are modelled as elastic members. This latter choice is made to compare the internal forces developed during the nonlinear analysis to the internal forces predicted in the phase of design.
The Rayleigh formulation is used to introduce damping. Mass and stiffness proportional damping coefficients are defined so that the first and the third modes of vibration of the system are characterised by an equivalent viscous damping ratio equal to 0.03. The stiffness proportional damping coefficient is applied to the committed stiffness matrix of the elements, except for the dissipative members of the CBFs (inelastic braces) where damping is not considered. \( P-\Delta \) effects are included in the numerical analyses.

5.2 Response parameters

Attention is focused on the ductility demand to capacity ratio of braces and on stability and resistance indexes of non-dissipative members.

The total ductility capacity of the brace at failure under the cyclic axial loading is calculated according to the formulation proposed by Marino,\(^4\) that is,

\[
\mu_f = 1 + \frac{\theta_f^2 E}{2f_y^2},
\]

where \( E \) is the elastic modulus of steel and \( \theta_f \) is the rotation at fracture, calculated under the assumption that the deformed shape of the buckled braces approaches that of a rigid stick model with a plastic hinge at the mid-length of the brace. Based on the results of cyclic experimental tests described in,\(^{24}\) \( \theta_f \) can be expressed as a function of the brace slenderness \( \lambda \), width-to-thickness ratio (\( b_0/t \)) and depth-to-thickness ratio (\( d_0/t \)), that is,

\[
\theta_f = 0.091 \left( \frac{b_0}{t} \right)^{0.1} \left( \frac{d_0}{t} \right)^{0.3}. \quad (19)
\]

Consistently with the definition of the total ductility capacity, for each accelerogram, the total ductility demand \( \mu_d \) of the braces is determined as the sum of ductility levels reached in tension and compression before fracture, that is, as the ratio of the sum of absolute values of the maximum elongation and shortening to the elongation at yield. Then, the damage index \( DI \) of each brace is calculated as a function of the ductility demand \( \mu_d \) and ductility capacity \( \mu_f \) of braces by means of the following relation:

\[
DI = \frac{\mu_d - 1}{\mu_f - 1}. \quad (20)
\]

A value of DI equal to unity corresponds to the achievement of ductility capacity of brace at fracture and thus to the NC limit state; instead, a value of DI equal to 0.75 is assumed to be representative of the achievement of the SD limit state. This value has been selected because, on average, the axial deformation capacities given by EC8-part 3\(^{23}\) for braces in compression and in tension at their SD limit state are about 0.75 times the corresponding capacities at the NC limit state.

Resistance (RI), lateral stability (SI), and lateral torsional stability (TI) indexes are determined for fragile members. In particular, RI is the maximum value of the ratios

\[
RI = \max \left\{ \frac{M_{Ed}(t)}{M_{N,Rd}(t)}; \frac{N_{Ed}(t)}{N_{pl,Rd}} \right\}, \quad (21)
\]

where \( M_{Ed}(t) \) and \( N_{Ed}(t) \) are the bending moment and the axial force at the generic instant \( t \) of the time history, \( N_{pl,Rd} \) is the plastic axial resistance and \( M_{N,Rd}(t) \) is the plastic flexural resistance reduced because of the axial force \( N_{Ed}(t) \). Lateral stability (SI) and lateral torsional stability (TI) indexes are calculated as

\[
\begin{align*}
\frac{N_{Ed}(t)}{N_{pl,Rd}} + k_{yy} \frac{M_{Ed,y}(t)}{\chi_{y,pl,Rd,y} M_{pl,Rd,y}} + k_{yz} \frac{M_{Ed,z}(t)}{\chi_{z,pl,Rd,z} M_{pl,Rd,z}} & \leq 1, \\
\frac{N_{Ed}(t)}{N_{pl,Rd,z}} + k_{xy} \frac{M_{Ed,y}(t)}{\chi_{y,pl,Rd,y} M_{pl,Rd,y}} + k_{zz} \frac{M_{Ed,z}(t)}{\chi_{z,pl,Rd,z} M_{pl,Rd,z}} & \leq 1,
\end{align*}
\]

\( (22) \)
where \( N_{b,Rd,y} \) and \( N_{b,Rd,z} \) are the buckling resistances with respect to the strong and the weak axes, \( \chi_{LT} \) is the reduction factor due to lateral torsional buckling and \( k_{yy}, k_{zz}, k_{yz}, k_{zy} \) are interaction factors accounting for the slenderness of the members and the shape of the bending moment diagram. When SI is calculated, \( \chi_{LT} \) is set equal to 1.0 and the interaction factors are determined as reported in table B.1 of Annex B of Eurocode 3.\(^{[25]}\) When TI is determined, \( \chi_{LT} \) is calculated as reported in Eurocode 3 (6.3.2) and the interaction factors are determined as reported in table B.2 of Annex B of Eurocode 3.

Referring to the single ground motion, the PGA corresponding to the achievement of SD limit state is the minimum of the values of PGA leading to either \( DI = 0.75 \) in braces or to SI, RI and TI = 1.00 in columns or beams. This value of PGA is hereinafter defined as \( \text{PGA}^{d,c,b}_{\text{SD}} \). Suffixes \( d, c \) and \( b \) indicate that the estimate of the PGA is carried out considering diagonal members (‘d’), columns (‘c’) and beams (‘b’).

If the rules for the application of the capacity design principles were perfectly effective, braces should exploit their deformation capacity, while beams and columns should be in the elastic range of behaviour. Consequently, the value of \( \text{PGA}^{d,c,b}_{\text{SD}} \) should be equal to the value of PGA, leading to \( DI = 0.75 \) in braces (later on named \( \text{PGA}^{d}_{\text{SD}} \)). Conversely, if the rules for the application of capacity design principles are not effective, \( \text{PGA}^{d,c,b}_{\text{SD}} \) is smaller than \( \text{PGA}^{d}_{\text{SD}} \).

To identify cases in which the rules for the application of the capacity design principles fail in preventing yielding or buckling of columns or beams, other two reference values of the PGA are defined. Specifically, still referring to the selected ground motion, \( \text{PGA}^{d,c,b}_{\text{SD}} \) is the minimum PGAs leading to either \( DI = 0.75 \) or SI, RI and TI = 1.00 in columns, whereas \( \text{PGA}^{d,c,b}_{\text{NC}} \) is the minimum of the PGAs leading to either a \( DI = 0.75 \) or SI and RI = 1.00 in beams. Finally, for each reference PGA (\( \text{PGA}^{d,c,b}_{\text{SD}}, \text{PGA}^{d,c,b}_{\text{SD}}, \text{PGA}^{d,c,b}_{\text{SD}}, \text{PGA}^{d,c,b}_{\text{SD}} \)), the average value over the number of considered accelerograms is calculated.

Similarly, four average values of PGAs are determined at the achievement of NC limit state (\( \text{PGA}^{d,c,b}_{\text{NC}}, \text{PGA}^{d,c,b}_{\text{NC}}, \text{PGA}^{d,c,b}_{\text{NC}}, \text{PGA}^{d,c,b}_{\text{NC}} \)). However, note that in this case a limit value of \( DI = 1.00 \) is assumed for braces.

Finally, to evaluate effectiveness of the considered design procedures in predicting the internal forces in columns, attention is also focused on the axial forces and bending moments in these members.

### 6 EFFECTIVENESS OF OVERSTRENGTH MEASURES TO PROMOTE UNIFORM DAMAGE

All the considered design methods are intended to promote a ductile collapse behaviour by means of the verification of an almost uniform distribution of overstrength factor of braces (\( \Omega_{\text{max}}/\Omega_{\text{min}} \leq 1.25 \)). These methods, however, differ because of the evaluation of this overstrength factor. Specifically, the EC8 and CDL design methods prescribe that the brace overstrength factor be calculated as the ratio of the buckling resistance to the design axial force of the brace, whereas the MN design method suggests that the overstrength factor be calculated as the ratio of the storey shear resistance corresponding to the sum of the plastic resistance of the brace in tension and the postbuckling resistance of the brace in compression to the design storey shear force. To evaluate the effectiveness of these definitions of the overstrength factor and to give voice to the belief of other researchers that argue that the overstrength factor should be calculated as the ratio of the plastic resistance of the brace to the design axial force in the brace, the overstrength factor of the single storey is computed for each designed case study as the ratio of the storey shear resistance to the design storey shear force. The storey shear resistance is provided by either the buckling resistance of the brace, the sum of the plastic resistance of the brace in tension and the postbuckling resistance of the brace in compression, or the plastic resistance of the brace. The ratios \( \Omega_{\text{max}}/\Omega_{\text{min}} \), \( \Omega_{\text{max}}/\Omega_{\text{min}} \), and \( \Omega_{\text{max}}/\Omega_{\text{min}} \) of the maximum to minimum overstrength factors, respectively, calculated according to the above definitions are plotted against the average damage index \( \text{DI}_{\text{m}} \) of the braces. The brace damage considered for the single ground motion is calculated on the occurrence of PGA leading to the NC limit state in braces, that is, the \( \text{PGA}^{d}_{\text{NC}} \), of the single ground motion. Hence, any premature conventional collapse caused by yielding or buckling of fragile members is neglected; that is, only the achievement of the damage index \( DI = 1.0 \) in braces is considered. In addition, when the CDL design procedure is applied, the top storey braces are not considered in the evaluation of the average damage index \( \text{DI}_{\text{m}} \) because such members are designed to remain elastic.

Note that values of \( \text{DI}_{\text{m}} \) equal to 1.0 are representative of very ductile collapse mechanisms characterised by the full exploitation of the ductility capacity of braces at each storey, while low values of \( \text{DI}_{\text{m}} \) are proper of collapse mechanisms with scattered distribution of damage. The results of above-described investigation are plotted in Figure 3, where the single dot refers to the single structure designed by means of either the EC8, MN or CDL design method. The ordinate of the dot represents the average damage \( \text{DI}_{\text{m}} \) of the braces, while the abscissa is equal to the ratio \( \Omega_{\text{max}}/\Omega_{\text{min}} \) computed
Figure 3 shows the damage index of braces as a function of the ratio of the maximum to the minimum storey overstrength factor calculated by means of (A) buckling resistance of braces, (B) sum of plastic and postbuckling resistances of braces and (C) plastic resistance of braces. According to either of the definitions reported above, a measure of the overstrength is considered to be effective if the smaller the overstrength ratio, the larger the average damage index; that is, if the Pearson correlation coefficient is calculated, a negative value should be obtained.

Figure 3A shows the average damage DIm as a function of scattering of overstrength factor \((\Omega_{\text{max}}/\Omega_{\text{min}})_{\text{calc}}\) as a function of the sole buckling resistance of braces. In this case, the average damage of braces DIm decreases with the increase of the ratio \(\Omega_{\text{max}}/\Omega_{\text{min}}\) and is almost constant for very scattered distributions of the overstrength factor. This leads to a value of the Pearson coefficient equal to −0.35. In Figure 3B, the ratio \(\Omega_{\text{max}}/\Omega_{\text{min}}\) is calculated as a function of the sum of the plastic and postbuckling resistances of braces. If the Pearson correlation coefficient is calculated, a positive value is obtained, meaning that the damage index of braces DIm is larger for structures with scattered overstrength. Finally, Figure 3C considers the ratio \(\Omega_{\text{max}}/\Omega_{\text{min}}\) calculated as a function of the plastic resistance of braces and, again, a positive value of the Pearson correlation coefficient (0.31) is achieved.

In view of these observations, the choice of the buckling resistance of braces for the calculation of the overstrength factor appears to be the most adequate to be applied as it is the only that ensures high values of the average damage index of braces in the occurrence of low scattered overstrength factors.

7 EFFECTIVENESS OF RULES FOR APPLICATION OF CAPACITY DESIGN PRINCIPLES IN COLUMNS

Columns of braced frames are designed in modern seismic codes according to capacity design principles and thus buckling or lateral torsional buckling of these members should be prevented up to the achievement of the ductility capacity of braces. In addition, EC8 requires that columns remain in the elastic range of behaviour.

To comment on the effectiveness of rules proposed in the considered design procedures for application of the capacity design principles, the obtained values of \(\text{PGA}_{\text{SD}}^{\text{EC8}}\) and \(\text{PGA}_{\text{SD}}^{\text{MN}}\) for the examined structures are compared (Figure 4). In
particular, the top of the grey bars represents the value of $\text{PGA}_{\text{d,c}}$, while the top of the white bars represents $\text{PGA}_{\text{d}}$. Note that values of the ratio $\frac{\text{PGA}_{\text{d,c}}}{\text{PGA}_{\text{d}}}$ equal to unity, that is, null values of the white bars, are indicative of very effective rules. Indeed, such a result points out that on the occurrence of each accelerogram, the SD limit state is achieved because of a damage index $\text{DI}$ of braces equal to 0.75. On the contrary, values of the above ratio much lower than unity are indicative of ineffective rules as they prove that the SD limit state has been prematurely achieved because of buckling or yielding of columns. A comparison similar to that described above is also carried out with reference to $\text{PGA}_{\text{d,c}}$ and $\text{PGA}_{\text{d}}$ to distinguish in which cases the NC limit state is caused by the achievement of a damage index $\text{DI}$ equal to 1.00 from those in which the same limit state is mostly caused by buckling or yielding of columns. These results are illustrated in Figure 5.

If buckling and yielding of non-dissipative members are ignored, that is, if $\text{PGA}_{\text{d}}$ and $\text{PGA}_{\text{d,NC}}$ are evaluated, structures designed according to the CDL procedure generally reach the SD limit state for PGAs larger than the design value (0.35$g$); on the contrary, structures designed according to MN procedure reach the SD limit state for PGAs significantly lower than 0.35$g$. This behaviour is due to the significant concentration of damage in braces of single storeys because of the adopted measure of the brace overstrength. An intermediate performance is obtained in structures designed according to EC8. In regard to the achievement of their NC limit state, it should be noted that no prescription is given in EC8 for the minimum PGA required in new buildings. However, the NC limit state should be verified in existing buildings on the occurrence of seismic events characterised by a probability of exceedance equal to 2% in 50 years. According to the Italian Seismic Code, instead, the NC limit state should be verified under seismic events characterised by a probability of exceedance of 5% in 50 years. These different statements of the European and Italian seismic codes lead to the requirement of a minimum PGA at the achievement of NC limit state in existing buildings equal to 0.60$g$ and 0.45$g$, respectively. Figure 5 shows that the above observations given with regard to the SD limit state also apply to the NC limit state if a reference value of 0.45$g$ is considered for the PGA. Instead, only one 12-storey structure satisfies the requirement of EC8 with regard to the NC limit state; that is, only one structure is characterised by a $\text{PGA}_{\text{d,NC}}$ larger than 0.60$g$. Figure 5 also underlines that, in the case of low-storey structures, the NC limit state is generally achieved under lower values of PGAs and thus a lower value of the behaviour factor should be used for the design of such systems.

The comparison of values of $\text{PGA}_{\text{d,c}}$ versus $\text{PGA}_{\text{d}}$ and $\text{PGA}_{\text{d,c}}$ versus $\text{PGA}_{\text{d,NC}}$ highlights that in no case the values are equal; that is, the SD and NC limit states are achieved because of yielding or buckling of columns on the occurrence of at least one accelerogram. The height of the white bars (i.e., the difference between the two values of PGA) is significant in the case of structures designed according to EC8. Some yielding or buckling of columns also occurs in structures designed by the CDL procedure, even though this latter procedure considers bending moments in columns. Smaller differences of PGA are generally obtained for structures designed according to the MN design procedure, particularly in the case of structures belonging to groups #1 and #2.

To investigate the reasons of the unsatisfactory behaviour of the columns, the maximum values of axial forces and bending moments obtained by the time history analysis under accelerograms scaled to $\text{PGA}_{\text{d,NC}}$ are compared to the internal forces predicted by the design method of analysis. Figure 6 illustrates the results obtained for the eight-storey structure of group #1 designed according to the three considered procedures. In particular, when the MN design method is used, axial forces in columns appear to be well predicted at the upper storeys and are slightly overestimated at the lower storeys. This overestimation is due to the conservative design assumption affirming that braces are simultaneously in the nonlinear range of behaviour at all storeys of the building. The design method of EC8 underestimates axial forces
in columns, particularly at the upper storeys. Indeed, the application of Equation (4) to the top storey columns provides axial forces caused by gravity loads only. The CDL design procedure, instead, considers two relations for the evaluation of the design internal axial forces of columns. The axial forces of the columns of the lower storeys are overestimated when Equation (4) (with the effect of seismic action amplified by $\Omega_{pl}$) is used and well predicted by values provided by Equation (12), whereas both the above-mentioned equations underestimate the axial forces at the upper storeys.

To investigate the reasons of this underestimate, Figure 7 shows for all the structures designed according to the CDL procedure the ratio of the axial forces caused in the top storey braces by nonlinear time history analysis ($N_{THA}^{Ed}$) to the corresponding axial forces provided by the response spectrum analysis ($N_{MRSA}^{Ed}$). Results referring to the single structure are represented by a grey bar; the lower bound, upper bound and middle of such a bar represent the minimum, maximum and average values of the examined response parameter on the occurrence of the considered 10 accelerograms. The figure shows that all the braces of the upper storey are elastic (they are designed to sustain an axial force equal to the behaviour factor $q$ times $N_{MRSA}^{Ed}$) and, on the average, the ratio $N_{THA}^{Ed}/N_{MRSA}^{Ed}$ is close to $q/2 = 2.0$. Thus, the axial force of the top storey braces should not be neglected in the design phase when axial forces of columns of the upper storeys are evaluated.
Figure 6 also shows that columns are subjected to very high values of the bending moments. These bending moments cannot be predicted by the design method of analysis because, when braces are elastic, the lateral displacement profile due to the seismic force is almost linear and thus columns basically rotate rigidly about their base. Conversely, after buckling of the brace in compression and yielding of the brace in tension at a single storey, the lateral storey stiffness decreases significantly and the lateral displacement profile changes accordingly. In this phase of the structural response, columns develop large bending moments. At achievement of the NC limit state these bending moments are even larger than 0.2 $M_{pl,Rd}$. As a consequence of the unconservative prediction of both the bending moments and the axial forces in columns, yielding and buckling occur in columns prior to the achievement of the ductility capacity of braces. Interested readers can find a detailed discussion on the response of frame EC8-08-#1 at the website http://www.dica.unict.it/users/mbosco/VCBF-CapacityDesign.htm

8 EFFECTIVENESS OF RULES FOR APPLICATION OF CAPACITY DESIGN PRINCIPLES IN BEAMS

To comment on the effectiveness of the rules for application of the capacity design principles in beams, $PGA_{SD}^d$ and $PGA_{SD}^{d,b}$ are compared along with $PGA_{NC}^d$ and $PGA_{NC}^{d,b}$ (Figure 8). When the EC8 design procedure is applied (first column of the figure), the unbalanced vertical force transferred by the tension and compression braces to the beam is significantly underestimated. As a consequence, yielding of the beam often occurs for values of PGA lower than those leading to the full exploitation of the ductility capacity of braces. The ratios $PGA_{NC}^{d,b}/PGA_{NC}^d$ and $PGA_{SD}^{d,b}/PGA_{SD}^d$ are generally decreasing with the increase in the number of storeys. The more stringent requirements of the CDL design procedure (third column of the figure) enhance the performance of the beams with respect to that of the same members in the structures designed according to EC8. Note that in the structures designed by the CDL procedure and belonging to groups #1 and #3 (i.e., in the structures with braces characterised by steel grade S355), the design of the beams at the lower storeys is governed by the strength requirement, while that of the beams at the other storeys is governed by the stiffness and strength requirements; in the structures belonging to groups #2 and #4 the design of the beams is always governed by the stiffness requirement. Thus, independently of the accuracy in the evaluation of the design bending moments, yielding of beams of buildings
belonging to groups #2 and #4 is prevented because of the overstrength caused by the stiffness requirement. In case of the structures designed according to the MN procedure, instead, yielding of the beams never occurs owing to the accurate prediction of the axial force of braces in their postbuckling range of behaviour.

Finally, the authors underline that a moderate yielding of braced beams could be not indicative of collapse provided that width-to-thickness ratios of the cross-sections are adequate and that the axial forces in beams do not reduce the deformation capacity of these members. In the three design procedures considered in this paper, however, no specific provision is given to ensure a moderate ductility capacity of braced beams.

9 | PROCEDURE PROPOSED TO PREDICT THE BENDING MOMENTS IN COLUMNS

Previous numerical analyses show that all the examined design procedures substantially neglect or significantly underestimate the bending moments in the columns of the braced frames. These bending moments \( M_{Ed,c} \) depend on the interstorey displacements and on the flexural rotations of the end nodes of columns. The following relation, obtained by a slight modification of the one proposed in\(^{18} \) for suspended zipper-braced frames, is suggested here to estimate the bending moment of the column at the \( i \)-th storey at the achievement of ductility capacity of braces at the same storey:

\[
M^i_{Ed,c} = \rho_n \kappa^i \rho^s_i \frac{E I^i_c}{H^i} u^i_{f,br} \quad i = 1, 2, \ldots, n, \tag{23}
\]

where \( n \) is the number of storeys, \( h \) is the interstorey height, \( I^i_c \) is the moment of inertia of the column cross-section, \( E \) is the elastic modulus of steel and \( u^i_{f,br} \) is the interstorey displacement corresponding to the achievement of the ductility capacity of brace.

The parameter \( \rho_n \) in the above relation is introduced because the maximum bending moment is generally caused by lateral displacements corresponding to the higher modes of vibration and thus the distance between successive contraflexure points in the lateral deformation of column depends on the number of storeys. This parameter is calculated in this paper by means of the relation

\[
\rho_n = \frac{10}{n} \leq 1. \tag{24}
\]

The parameter \( \kappa^i \) considers that the flexural stiffness at the ends of the column at the \( i \)-th storey is not equal along the height of the frame because of the presence of a pin connection at the base of the column and is calculated in this study as

\[
\kappa^i = 1.2 \cdot \left( 0.7 + 0.3 \sqrt{\frac{i}{n}} \right). \tag{25}
\]

Still in Equation (23), the parameter \( \rho^s \) is introduced to consider that the bending moments in the columns also depend on the ratio of the lateral stiffness of braces to the lateral stiffness of columns. In this paper, \( \rho^s \) is calculated at each storey as

\[
\rho^s_i = \frac{1}{\rho_n} \sqrt{\frac{1}{50} \sum \frac{A_{br}}{L_{br}} \cos^2 \alpha \cdot \frac{h^3}{\sum I_c}} \leq 1, \tag{26}
\]

where \( A_{br} \) is the cross-section area of the brace, \( \alpha \) is the angle of the brace measured with respect to the horizontal axis, and \( L_{br} \) is the length of the brace. In the same relation, the summation is extended to all the braces or columns (including both gravity and seismic columns) of the storey under examination. The interstorey displacement corresponding to the achievement of the ductility capacity of braces can be calculated as

\[
u^i_{f,br} = \frac{L_{br} \delta^i_{ui}}{L + 2v_{bs} \sin \alpha \cdot \cos \alpha}, \tag{27}\]
where $\delta_u$ is the axial displacement capacity of the brace and $v_b$ is the vertical displacement produced at the mid-span point of the beam by the unbalanced vertical force transferred by braces, that is,

$$
v_b = \gamma_{ov} \frac{(1.1N_{pl,br} - N_{u,br}) \sin \alpha \cdot L^3}{48EI_b},
$$

where $I_b$ and $L$ are the moment of inertia and the length span of the beam.

Equation (27) has been derived based on the following three conditions. First, the sum of the absolute values of the elongation $\delta_1$ of brace in tension and the shortening $\delta_2$ of brace in compression are equal to the axial displacement capacity $\delta_u$ of the brace, that is, the elongation at yield $\delta_y$ times the total ductility of the brace at fracture $\mu_f$. Second, the axial deformations $\delta_1$ and $\delta_2$ are related to the horizontal displacement $u_{f,br}$ and to the vertical displacement $v_b$, as given by the relations

$$
\delta_1 = u_{f,br} \cdot \cos \alpha - v_b \cdot \sin \alpha,
$$

$$
\delta_2 = u_{f,br} \cdot \cos \alpha + v_b \cdot \sin \alpha.
$$

Third, the point of intersection of the two braces is the intersection of two circumferences: the first circumference has radius $R_1 = L_{br} + \delta_1$ and is centred at the base of the left column, whereas the second has radius $R_2 = L_{br} - \delta_2$ and is centred at the base of the right column.

It is worth noting that Equation (23), proposed to predict bending moments in columns, assumes that the interstorey displacement reaches the value corresponding to the assumed deformation capacity of braces. Thus, this relation is expected to provide conservative values of the bending moments at storeys where the damage index of braces is lower than unity. The effectiveness of the proposed equation in the estimation of bending moments is shown in Figure 6 for the eight-storey structure of group #1 designed according to the three considered procedures. The comparison between predicted bending moments and values obtained by the numerical analyses shows a satisfying accuracy of the proposed equation. Further, even if not shown in any figure, the higher the number of storeys, the better the accuracy in predicting the bending moments.

A second requirement in the design of columns should be added to promote a collapse mechanism characterised by a virtually full exploitation of the ductility capacity of braces at all storeys. In particular, reduction of the lateral storey stiffness that occurs when braces of a single storey enter the plastic range of behaviour should be limited by calibration of the flexural stiffness of columns and thus of the value of the parameter $\rho_s$. The maximum value of $\rho_s$ to be adopted is calibrated in the following section.

### 10 EFFECTIVENESS OF PROPOSED EQUATION IN PREVENTING BUCKLING OF COLUMNS AND CALIBRATION OF STIFFNESS REQUIREMENT OF COLUMNS

To assess the effectiveness of Equation (23) in predicting the bending moments of columns at the NC limit state, the gravity columns and the columns of the braced frames of all the structures described in Section 3 have been redesigned based on the values of the bending moments predicted by the proposed relation.

In a first stage of this investigation, when possible, only the strength of the column cross-section is increased, so as to keep $\rho_s$ as close as possible to the value obtained by the initial design. The obtained structures are identified by suffix ‘RoInf’ to underline that no stiffness requirement is considered, that is, no upper limit of $\rho_s$ is fixed.

The design of the columns of these structures is generally governed by the lateral torsional buckling verification. In this regard, the authors note that in the design phase, columns are considered as members with sway buckling mode. Thus, according to Annex B of Eurocode 3,25 the equivalent uniform moment factor, which takes into account the shape of the bending moment diagram, is set equal to 0.9.

Then, to calibrate the maximum value of $\rho_s$ to be suggested, later named $\rho_{s,\text{max}}$, the stiffness of the columns is also verified to limit the obtained values of $\rho_s$ to a maximum equal to either 1.2 (structures ‘Ro120’) or 1.0 (structures ‘Ro100’). Wherever the column cross-section is increased to satisfy the stiffness requirement, a steel grade with moderate yield
strength (S235 or S275) is tentatively used to avoid significant overstrength in such columns. In the case of structures obtained from the ones designed according to the CDL procedure, the condition on the maximum value of $\rho_s$ is not forced at the top storey of the building, because at this storey the braces are designed to be elastic.

To evaluate the structural costs due to the proposed design modifications, the structural costs of the columns is determined in all cases and compared. The assumed unit price refers to the Sicilia Region’s official price list. In particular, the cost of H-shaped steel cross-sections is equal to 3.53€/kg for steel grade S235 or S275 and 3.73€/kg for S355. The obtained column cost $C_c$ is normalised to the number of storeys and to the floor area ($24 \times 24$ m) and plotted by coloured bars in Figure 9 for the cases obtained from the standard formulation of the examined (EC8, MN and CDL) design procedures (named ‘noM’ in figures) and for all the cases obtained from these systems by modification according to the proposed design conditions (cases named RoInf, Ro120 and Ro100). Moreover, a solid black line is added in the plots to point out the percentage variation of the column costs caused by the proposed procedure with respect to the standard application of the examined design procedure. As is evident, the average cost per square metre of the columns is increasing with the number of storeys and is larger in buildings belonging to groups #1 and #2 because of the larger gravity loads and interstorey height. Further, it is no doubt that the average cost of the columns increases with the decrease of the maximum value considered in design for the parameter $\rho_s$.

As a general trend, the increase caused by the proposed procedure in the structural cost of columns (see the solid lines) is larger in the four-storey buildings than in eight- or 12-storey buildings. This increase is generally significant for
FIGURE 10  Effect of design provisions on PGAs corresponding to the achievement of SD limit state in structures previously designed according to (A) EC8, (B) MN or (C) CDL design procedure

The results referring to the seismic response of the newly designed structures at the achievement of the SD and NC limit states are plotted in Figures 10 and 11, respectively. The PGAs corresponding to the achievement of the above limit states, that is, $\text{PGA}_{\text{SD}}^{d,c}$ or $\text{PGA}_{\text{NC}}^{d,c}$, are represented by coloured bars. The values of $\text{PGA}_{\text{SD}}^{d}$ and $\text{PGA}_{\text{NC}}^{d}$ are also reported on the same graph and indicated by the top of the white bars. The height of the white bars allows a direct evaluation of the effects of rules for the application of capacity design principles in the columns. Figure 10 points out that a proper estimation of the bending moments in columns (RoInf in the figure) appreciably improves the effectiveness of the capacity design rules in columns and also reveals the most significant effects in buildings with a high number of storeys. In the structures designed according to the above modification, the values of $\text{PGA}_{\text{SD}}^{d} - \text{PGA}_{\text{SD}}^{d,c}$ are close to zero, except for a few exceptions represented by cases CDL-04#02, CDL-08#02, CDL-12#01 and CDL-12#02. In these latter structures, the limit state is prematurely achieved by lateral torsional buckling of the second top storey columns owing to underestimation of the axial force in these members.
The reduction in values of the differences between $\text{PGA}_{\text{SD}}^d$ and $\text{PGA}_{\text{SD}}^{d,c}$ from those corresponding to the standard application of the design procedures to the values corresponding to the suggested estimation of the bending moments in columns (RoInf in the figure) is higher in the case of application of EC8 and CDL design procedures. This is the result of the combined effect of underestimation of the axial force in columns and the less effective prediction of the bending moments in the standard formulations of the design procedures. The enforcement of a maximum value of $\rho_s$ equal to 1.20 (Ro120 in the figure) has slight favourable effects on the reduction of $\text{PGA}_{\text{SD}}^d - \text{PGA}_{\text{SD}}^{d,c}$ of most structures, except for a few cases initially designed according to the CDL design procedure, where the limitation of the lateral deformability of columns causes a significant reduction of the height of the white bars. Indeed, the application of the above additional design prescription to these few cases leads to an increase in their second top storey column cross-section where premature buckling of columns occurred. The same design modification has an impact on the values of $\text{PGA}_{\text{SD}}^d$ in the structures designed according to the EC8 and MN procedures, because the distribution of damage in the structures initially designed according to these design procedures was scattered; no significant increase in the values of $\text{PGA}_{\text{SD}}^d$ is recorded for structures designed according to the CDL procedure, because the distribution of damage in the structures was less scattered. No significant changes are achieved when enforcing a more severe limit on the value of $\rho_s$ equal to 1.0. Similar considerations can be drawn on the basis of Figure 11, where the PGAs are plotted at the achievement of the NC limit state. Referring to this figure, it is only to note that in a low number of cases the height of white bars is not null, even in the case of application of $\rho_{s,\text{max}}$ equal to 1.0. The above number of cases is, however, very low and does not justify, in the opinion of the authors, the use of values of $\rho_{s,\text{max}}$ equal or even lower than unity, standing the implications on the costs of the structures. Based on these considerations, a value of $\rho_{s,\text{max}}$ equal to 1.2 is suggested here.
FORMULATION OF PROPOSED DESIGN PROCEDURE

The analysis of response of the structures investigated in the previous sections highlights strengths and weaknesses of the considered design procedures. Based on this, a force-based design procedure is proposed here and applied according to the following steps:

1. Axial forces on braces are determined by modal response spectrum analysis. The design spectrum is obtained by means of a behaviour factor equal to 4.0 for structures with a number of storeys not less than eight. Based on the results of a parametric analysis not reported in this paper, a value of the behaviour factor equal to 3.0 is suggested for structures with a number of storeys not higher than four. This ensures values of PGA leading to the SD and NC limit states that are almost independent of the number of storeys of the structure; linear interpolation can be applied to obtain behaviour factors for structures with a number of storeys between four and eight.

2. The brace cross-section and steel grade are selected so that the buckling resistance is higher than the design axial force. In keeping with the CDL design procedure, top storey braces are designed to remain elastic in the seismic design situation and thus to sustain an axial force equal to value obtained by the above modal response spectrum analysis multiplied by the behaviour factor. To promote a dissipative behaviour of the structure, the brace overstrength factor is calculated as the ratio of the buckling resistance to the design axial force of the brace, and the ratio of the overstrength factor to the minimum overstrength (\( \Omega_{\text{min}} \)) has to be lower than 1.25. In this regard, the authors note that the overstrength factor is not to be calculated at the top storey because at this storey the braces are supposed to be elastic.

3. The axial force of the columns of the top storey is given by the gravity loads only, whereas the axial forces of the columns of all the other storeys are determined by equilibrium conditions (Figure 12) considering two different design situations derived from those stipulated by the MN design procedure. In the first situation, the axial forces of the braces in tension and compression at the storeys from the first to the second top are equal to the buckling resistance; in the second design situation, at the storeys from the first to the second top, the brace in tension is yielded and fully hardened whereas the brace in compression is in its postbuckling range of behaviour. In both the design situations, the axial forces in the braces of the top storey are supposed to be equal to \( \psi N_{\text{MRSA}}^{\text{Ed}} \), where \( N_{\text{MRSA}}^{\text{Ed}} \) is the axial force obtained by the modal response spectrum analysis and \( \psi \) is given by the relation below

\[
\psi_i = \frac{q}{2} \left[ 1 + \frac{i - (n - 1)}{4} \right] \geq 0.
\]  

The design axial force of the columns is thus obtained by means of the relation

\[
N_{\text{Ed,c,i}} = N_{\text{Ed,G,i}} + \gamma_{\text{ov}} \max \left\{ \sum_{j=1}^{i-1} \frac{1.1N_{\text{pl,br,j}} - N_{u,br,j}}{2} \sin \alpha + \sum_{j=i+1}^{n-1} N_{\text{u,br,j}} \sin \alpha; \sum_{j=i+1}^{n-1} N_{b,br,j} \sin \alpha \right\} + \psi_i N_{\text{MRSA}}^{\text{Ed,i}} \sin \alpha.
\]  

(32)
The proposed expression of the coefficient $\psi$ is derived so that the contribution provided by the axial force of top storey braces gradually reduces from the $n$-th storey of the building (where $N_{Ed,br} = q/2N_{Ed}^{MRSA}$, i.e., the value obtained on average in Figure 7) to the $n$-th storey where it is assumed to be negligible. This gradual reduction is introduced to limit the overestimation of the axial force at the base of the building caused by the conservative hypothesis that assumes that the maximum forces in braces are reached simultaneously at all storeys of the building. Cross-sections of columns shall be selected to sustain axial forces provided by Equation (32) combined to bending moments determined by Equation (23).

Further, the lateral stiffness of columns shall be selected so as to have a maximum value of $\rho_s$ equal to 1.2.

1. The beam cross-section has to verify both strength and stiffness requirements. The beam cross-section should be able to sustain (i) the combined effect of axial force (derived by Equation 9), shear force and bending moments (derived by Equation 10); (ii) the effects of gravity loads without contribution of braces. Further, the moment of inertia of the beam cross-section should be selected so that the vertical displacement $v_b$ produced at mid-span by the unbalanced vertical force transferred by braces should be such that at the achievement of the ultimate interstorey displacement $u_{f,br}$, the elongation of the brace in tension $\delta_1$ be larger than 0.35 times the axial displacement capacity of the brace $\delta_u$. Based on Equations (27) and (30), the above condition, which promotes high dissipation of braces, leads to

$$v_b \leq -\frac{L_{br} + 2 \cdot 0.35 \cdot \delta_u}{2 \sin \alpha} + \frac{1}{2 \sin \alpha} \sqrt{L_{br}^2 + 4 \cdot (0.35 \cdot \delta_u)^2 + 2L_{br} \cdot \delta_u}. \quad (33)$$

12 SEISMIC RESPONSE OF STRUCTURES DESIGNED ACCORDING TO THE PROPOSED PROCEDURE

Structures belonging to groups #1 to #4 and characterised by a number of storeys equal to either four, eight or 12 are designed according to the proposed design procedure. The numerical model of these structures and the seismic input are those described in Section 5. The seismic response is determined by incremental nonlinear dynamic analysis, as applied in previous analyses. The PGAs corresponding to the achievement of the SD and NC limit states are summarised in Figure 13. Figure 13A shows that, with the sole exception of the four-storey structure belonging to group #2, the minimum value of PGA leading to the achievement of the SD limit state (top of the grey bars), that is, to either $DI = 0.75$ or max ($SI, RI, TI) = 1.00$ in columns or beams ($PGA_{SD}^{d,c,b} = PGA_{SD}^d$), is larger than the value adopted in design. Further, this limit state is always achieved because of the full exploitation of deformation capacity of the dissipative members (null height of the white bars, $PGA_{SD}^d = PGA_{SD}^{d,c,b} = 0$). Referring to the NC limit state, Figure 13B shows that, with the exception of the four-storey structure of group #2, the minimum value of PGA leading to either $DI = 1.00$ or max ($SI, RI, TI) = 1.00$ in columns or beams ($PGA_{NC}^{d,c,b} = PGA_{NC}^d$) is in the range from the value expected according to NTC18 to that expected in EC8. The NC limit state is obtained only for a very few accelerograms because of lateral torsional buckling of columns and this leads to some ratios $PGA_{NC}^{d,c,b}/PGA_{NC}^d$ slightly lower than 1.0 (height of the white bar larger than zero).

The obtained collapse mechanisms are characterised by fairly uniform heightwise distributions of the inelastic deformations of braces. Indeed, the average value of the damage index of braces at collapse ($DI_m$) is in the range from 0.45 to 0.75 (see Figure 14A). It should be noted that the improvement given by the proposed procedure is much higher than that arising from the comparison of Figures 14A and 3, because the values of $DI_m$ in Figure 3 were obtained neglecting the premature failure of the non-dissipative members.
Finally, the structural cost of the systems (including both members of the braced frames and members designed to sustain gravity loads) is determined. To this end, the cost of hollow steel cross-sections is assumed equal to 6.43€/kg for steel S235 or S257 and 6.74€/kg for S355, whereas the cost of H-shaped steel cross-sections is given in Section 10. The cost of the system is plotted in Figure 14B as normalised to the number of storeys and the floor area and it is measured on the vertical axis on the left side of the figure. In the same plot, the percentage increment of the structural cost with respect to that of the corresponding structures designed according to EC8, MN and CDL design procedures is also reported by solid lines with red squares, black triangles and white dots, respectively. The structural cost per square metre slightly increases with the number of storeys and is larger in groups #1 and #2 because of the higher vertical loads and interstorey heights. As is evident from the solid lines, the percentage increment of the structural cost with respect to that produced by the CDL design procedure (blue solid line with white dots) decreases with the increase of the number of storeys and is never higher than 15%. This increment is mostly given by the greater cross-section of the columns. The percentage increment of the structural cost with respect to that produced by the EC8 design procedure (red dashed line with red squares) does not show a specific trend but is always lower than 10%. This increase is produced by the greater cross-section of the columns and is almost independent of the number of storeys. Furthermore, it is moderate (and in some cases even a reduction of costs occurs) because of the reduction of the structural costs of braces due to the adoption of a behaviour factor higher than 2.5. The comparison of the structural costs with respect to that produced by the MN design procedure (dashed line with black triangles) shows a small reduction of the weight for the eight- and 12-storey structures of groups #2 and #4 and a small increase in all the other cases.

13 | CONCLUSIONS

The first part of the paper investigates the seismic response of a large number of buildings with concentric braces in the chevron configuration with pinned beam-to-column connections and designed according to three force-based design procedures, namely the procedure reported in the European seismic code, that proposed by Marino and Nakashima and that formulated by Costanzo et al. The investigated procedures differ because of the behaviour factor, the measure of the lateral storey strength and the rules for the application of the capacity design principles. The seismic performance of the structures is obtained by incremental nonlinear dynamic analyses by means of 10 artificially generated accelerograms. These numerical analyses lead to the following main conclusions:

- The requirement of an almost uniform distribution of the brace overstrength along the height of the building should be calculated based on the buckling resistance of braces in order to successfully promote a ductile collapse mechanism.
- The rules reported in the above design procedures for the application of the capacity design principles are generally not effective in avoiding buckling of columns. This result is striking when EC8 is applied, but also apparent in the case of the design procedure by Costanzo et al. and that by Marino and Nakashima, which better protect the frame from failure of columns. This unfavourable response is chiefly caused by the bending moments in columns, because these internal
actions are neglected in the procedures prescribed in EC8 and proposed by Marino and Nakashima and underestimated in the procedure formulated by Costanzo et al.

- The rules for the application of the capacity design principles reported in EC8 are generally not effective in avoiding yielding of braced beams. Indeed, the bending moment produced by the unbalanced vertical force transferred by braces is underestimated because the axial force of braces is overestimated in the postbuckling range of behaviour. Yielding of braced beams never occurs when the Marino and Nakashima design procedure is used while moderate yielding is obtained if the procedure proposed by Costanzo et al. is applied in structures with braces having high-strength steel.

Based on the results of the numerical analyses, a new relation is proposed to estimate the bending moments in columns, and a stiffness requirement is formulated for braced beams and columns in order to improve the global ductility of system. Based on these relations, a new design procedure is proposed and applied to a large number of CBFs. The analysis of the seismic response of these structures leads to the following conclusions:

- Buckling of columns and yielding of braced beams are avoided at the significant damage limit state and limited to a very few cases at the near-collapse limit state.
- The significant damage limit state is achieved for PGAs larger than the design value corresponding to a probability of exceedance of 10% in 50 years.
- The near-collapse limit state is achieved for PGAs that are in the range from that corresponding to a probability of exceedance of 5% in 50 years (as suggested in the Italian seismic code) to that corresponding to a probability of exceedance of 2% in 50 years (as suggested in EC8).
- The response corresponding to the achievement of the near-collapse limit state is characterised by an average value of the damage index of braces in the range from 0.45 to 0.75.
- The proposed procedure requires structural costs that, in the worst case, are only 10% higher than those required by the application of the other design procedures considered herein.

Finally, the authors note that numerical analyses have been carried out on numerical models in which beams and columns of the braced frames are elastic members. No significant change in the conclusions is expected if the columns were allowed to yield because these members are fragile due to the high values of the internal axial forces.

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**ORCID**
Francesca Barbagallo 🐦 https://orcid.org/0000-0001-5760-1681
Melina Bosco 🐦 https://orcid.org/0000-0002-6901-6612
Edoardo M. Marino 🐦 https://orcid.org/0000-0001-6127-1507
Pier Paolo Rossi 🐦 https://orcid.org/0000-0002-1680-0032

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