Behavior Factor of Dual Concentrically Braced Systems Designed by Eurocode 8

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SUMMARY:
In concentrically braced frames the low plastic redistribution capacity favours collapse configurations characterized by plastic deformations concentrated on a few storeys. The degrading character of the inelastic response of braces and the presence of scattered values of the storey overstrength factor further emphasize this behaviour. To improve the seismic response of these frames many researchers propose to couple concentrically braced frames with moment resisting frames. Despite the interest for the use of these dual structures the code procedures proposed for their design remain simplistic and not thoroughly investigated by researchers. To contribute to the knowledge of the effectiveness of these code procedures, several buildings with concentrically braced dual structure are designed here according to Eurocode 8. Their seismic response is obtained through incremental nonlinear dynamic analysis. The investigation aims to evaluate the correctness of the behaviour factor suggested in the Eurocode 8 and the effectiveness of the rules proposed for the application of the capacity design principles.

Keywords: Capacity design principles, behaviour factor, chevron configuration, nonlinear dynamic analysis

1. INTRODUCTION

Conventional concentrically braced frames (CBFs) are characterised by low plastic redistribution capacity and thus are prone to develop collapse configurations characterized by plastic deformations concentrated on a few storeys. The degrading character of the inelastic response of braces and the presence of scattered values of the storey overstrength factor further emphasize this behaviour. In the past, some researchers (Jain et al., 1993; Khatib et al., 1988) demonstrated that the seismic response of these systems can benefit from the interaction between moment resisting frames (MRFs) and braced frames. The moment resisting frames belonging to the dual system were first conceived as backup frames to the braced frame and thus were intended to provide strength and stiffness so as to prevent the collapse of the structure in the occurrence of an intense and rare ground motion (AISC, 2005). In accordance with this belief, seismic codes required that the braced frames were subjected to the whole seismic load and that the moment resisting frames were designed to resist seismic actions corresponding to the 25 per cent of the design base shear. Recently this conception has changed (Hines and Fahnestock, 2010) in that moment resisting frames are considered to be part of the primary lateral system. Some codes, e.g. Eurocode 8 (2005), agree with this new conception and require that dual structures are designed by means of a single behaviour factor and that the horizontal actions are distributed between the different frames according to their elastic stiffness.

Despite the potential benefits deriving from the adoption of a dual structure, design methods proposed by building codes for dual braced systems are simplistic and often not very effective. In this paper, the adequacy of the behaviour factor suggested in the Eurocode 8 for the design of dual systems consisting of chevron braced frames and moment resisting frames is investigated. Further, the effectiveness of the rules proposed for the application of the capacity design principles is critically discussed. To achieve this purpose, six dual structures are designed according to the procedure stipulated in Eurocode 8 and analysed by incremental nonlinear dynamic analysis.

2. DESIGN OF DUAL CHEVRON BRACED – MOMENT RESISTING FRAMES

Eurocode 8 reports only a few specific provisions for the design of dual braced structures. Also, these rules are addressed only to dual structures consisting of MRFs and CBFs. The procedure requires that the dual structure is designed by means of a single value of the behaviour factor and that the lateral forces are distributed between the CBFs and MRFs according to their lateral stiffness. The suggested behaviour factor is equal to 4 for systems with medium structural ductility (ductility class M) and equal to $4\alpha_u/\alpha_1$ for systems with high structural ductility (ductility class H). The parameter α is a multiplier of the seismic design actions: in particular, while the value α_u identifies the development of the overall structural instability, the value α_1 identifies the first yielding or buckling of members. These values of the behaviour factors are equal or slightly higher than those recommended for concentric active tension diagonal bracings but always significantly higher than those suggested for non-dual chevron braced structures. As behaviour factors suggested for structures consisting of different types of frames are expected to be slightly higher than the behaviour factors proposed for the single frames, it is reasonable to think that the behaviour factors proposed in the Eurocode 8 are intended for dual structures with concentric active tension diagonal bracings. Bearing in mind the logic of the provisions stipulated in the Eurocode 8, it is however reasonable to extend the area of applicability of the aforementioned rules and recommend that the behaviour factor of dual structures consisting of MRFs and braced frames in the chevron configuration should not be much higher than 2.5, i.e. not much higher than the value stipulated in the Eurocode 8 for non-dual CBFs in the chevron configuration.

Further, the Eurocode 8 recommends that the MRFs and the CBFs of the dual system are designed in accordance with the provisions stipulated for the two typologies independently considered. In regard to the application of the capacity principles to the aforementioned typologies, the dissipative zones of the structure are thus located in the diagonals of the CBFs and at the ends of all the beams, at the base of the first story columns and at the upper end of the top story columns of the MRFs. In accordance with the capacity design principles, while these dissipative zones are designed to resist the internal forces deriving from the seismic design situation, all the other parts are designed based on the strength of the dissipative zones.

2.1. Chevron braced frames

The minimum required value of the cross-sectional area of the braces A_b is obtained by equating the buckling resistance $N_{b,Rd}$ to the design axial force N_{Ed} of the brace in compression, as in the relation

$$A_b = \frac{\gamma_{M1}\left(N_{Ed,E} + N_{Ed,G}\right)}{\chi_b \quad f_y} \tag{2.1}$$

where γ_{M1} is the partial safety coefficient of steel, $N_{Ed,G}$ is the axial force due to the gravity loads in the seismic design situation, $N_{Ed,E}$ is the axial force due to the design seismic actions, f_y is the yield strength of steel and χ_b is the reduction factor for the buckling curve. In this investigation the latter parameter is calculated numerically (Bosco et al., 2009) and the partial safety coefficient γ_{M1} is assumed equal to 1. As recommended in the Eurocode 8, the buckling resistances of the braces are verified to satisfy the *homogeneity strength condition*. To accomplish this check, the brace overstrength factor Ω is first calculated at each storey of the building as the ratio of the buckling resistance $N_{b,Rd}$ to the design axial force N_{Ed} of the braces of the storey under investigation. Then, the maximum overstrength factor is verified not to be higher than 1.25 times the minimum value in the structure.

In regard to the seismic response of CBFs to design ground motions, the Eurocode 8 requires that braces dissipate energy while beams and columns remain elastic. According to this principle, the design internal forces of beams and columns should be obtained by equilibrium assuming that braces in tension are yielded while those in compression are in their post-buckling range of behaviour. In the Eurocode 8, instead, the above principle is applied through simplified rules. Specifically, the design axial force of beams and columns is given by the sum of two contributions: the axial force $N_{Ed,G}$ due to the gravity loads of the seismic design situation and the axial force $N_{Ed,E}$ due to the design seismic actions amplified by the coefficient $1.1\gamma_{ov}\,\Omega_{min}^{CBF}$, i.e.:

$$N_{Ed} = N_{Ed,G} + 1.1\gamma_{ov}\,\Omega_{min}^{CBF}\,N_{Ed,E} \tag{2.2}$$

In this relation, γ_{ov} is the overstrength of steel, i.e. the ratio of the average yield strength to the nominal value of the yield strength and Ω_{min}^{CBF} is the minimum overstrength factor of the braces.

The design capacity principles are applied strictly to evaluate the design bending moment on the beams of the braced frame. The bending moment due to the seismic actions is produced by the unbalanced vertical force transmitted by braces in their inelastic range of behaviour. This force gains the maximum value when the brace in tension has yielded and the brace in compression is in its post-buckling range of behaviour. In the European code the design value of this unbalanced force is calculated assuming that the axial force in the brace in tension is equal to $N_{pl,Rd}$ and that the axial force in the brace in compression is equal to the post-critical value $N_{u,Rd}$.. The same code recommends that the bending moment due to the non-seismic actions should be calculated assuming no support by the intermediate braces. Consequently, the design bending moment of the beam is calculated as

$$M_{Ed} = \left(N_{pl,Rd} - N_{u,Rd}\right)\mathrm{sen}\,\theta\,\frac{L}{4} + \frac{g_{Ed}L^2}{8} \tag{2.3}$$

where g_{Ed} is the distributed gravity load of the beam, θ is the angle of inclination of the brace with respect to the longitudinal beam axis and L is the length of the beam supposed to be pinned at both ends. The post-critical axial force $N_{u,Rd}$ is calculated numerically, as reported in Bosco et al. (2009).

Once the design axial forces and bending moments have been evaluated, the minimum required value of the modulus of resistance of the beam cross-section is calculated by equating the bending moment at mid-span to the flexural strength $M_{N,Rd}$ reduced by the axial force. The cross-sectional area of the columns is obtained by equating the design axial force of the columns to the buckling resistance of the same members. Both flexural and buckling resistances are determined according to Eurocode 3 (2005). The partial safety coefficients γ_{M0} and γ_{M1} are assumed equal to 1.

2.2. Moment resisting frames

The minimum required value of the modulus of resistance of the beams is obtained by equating the design bending moment to the plastic bending moment of the beams. Design bending moments are calculated by adding the effects of the gravity loads and seismic actions considered in the seismic design situation. As recommended in the Eurocode 8, 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 verified to sustain the gravity loads of the non-seismic design situation and to limit the deflection to the reference value reported in the Eurocode 8 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 $M_{pl,Rd}$ to the design bending moment M_{Ed}. The design internal forces of the non-dissipative zones of columns are obtained by adding the internal forces caused by the gravity loads of the seismic design situation to the internal forces caused by the design seismic forces amplified by the coefficient $1.1\gamma_{ov}\,\Omega_{\min}$:

$$N_{Ed} = N_{Ed,G} + 1.1\,\gamma_{ov}\,\Omega_{min}^{MRF}\,N_{Ed,E} \tag{2.4a}$$

$$M_{Ed} = M_{Ed,G} + 1.1\,\gamma_{ov}\,\Omega_{min}^{MRF}\,M_{Ed,E} \tag{2.4b}$$

$$V_{Ed} = V_{Ed,G} + 1.1\,\gamma_{ov}\,\Omega_{min}^{MRF}\,V_{Ed,E} \tag{2.4c}$$

The column cross-sections are selected so that two conditions are verified: (i) the design bending moment is lower than the flexural strength $M_{N,Rd}$ reduced by the axial force; (ii) the design axial force is lower than the buckling resistance reduced by the design bending moment. Both flexural and buckling resistances are calculated according to Eurocode 3 (2005).

2.3. Influence of second order effects

The strength required to counterbalance the second order effects (P-Δ effects) is estimated by means of the drift sensitivity coefficient θ evaluated as

$$\theta = \frac{P_{tot}\,\Delta u}{V_{Ed}\,h} \tag{2.5}$$

where P_{tot} and V_{Ed} are the total cumulative gravity load and the seismic shear at the storey under consideration; h is the interstorey height and Δu is the design storey drift, i.e. the elastic storey drift resulting from the design analysis multiplied by q.

P-Δ effects can be ignored if the drift sensitivity coefficient θ is everywhere lower than 0.1. These effects can be accounted for by means of a simplified approach if the coefficient θ is lower than 0.2; specifically, in these cases the Eurocode 8 suggests amplifying the internal forces resulting from the design seismic forces by means of the coefficient 1/(1- θ). No value of θ larger than 0.3 is accepted.

3. DESIGNED BUILDINGS

The design procedure described in Section 2 is applied to 4-, 8- and 12-storey buildings founded on soft soil (class C according to Eurocode 8). The plan is square-shaped (24 x 24 m^2) and the interstorey height h is equal to 3.3 m (Fig. 1). The geometric and mass properties of the buildings are equal at all storeys. Vertical dead and live loads are defined by characteristic values (G_k and Q_k) equal to 4.4 and 2.0 kN/m², respectively. In the non-seismic design situation (γ_g G_k + $\gamma_q Q_k$) the partial load safety factors γ_g and γ_q are assumed equal to 1.4 and 1.5. In the seismic design situation, the seismic actions are calculated on the basis of masses corresponding to a mean value of the gravity loads equal to 5.0 kN/m². The internal forces due to the design seismic actions are calculated by means of either the modal response spectrum analysis (MRSA) or the lateral force method of analysis (LFMA) on the basis of the elastic response spectrum proposed by the Eurocode 8 and scaled to a peak ground acceleration equal to 0.35 g. The design spectrum is obtained by reducing the elastic response spectrum by means of a behavior factor q equal to 2.5. In total, six buildings are designed (4-, 8- and 12-storey buildings designed by the MRSA or LFMA). The structural scheme is constituted by the intersection of two sets of four three-bay frames arranged along two orthogonal directions.

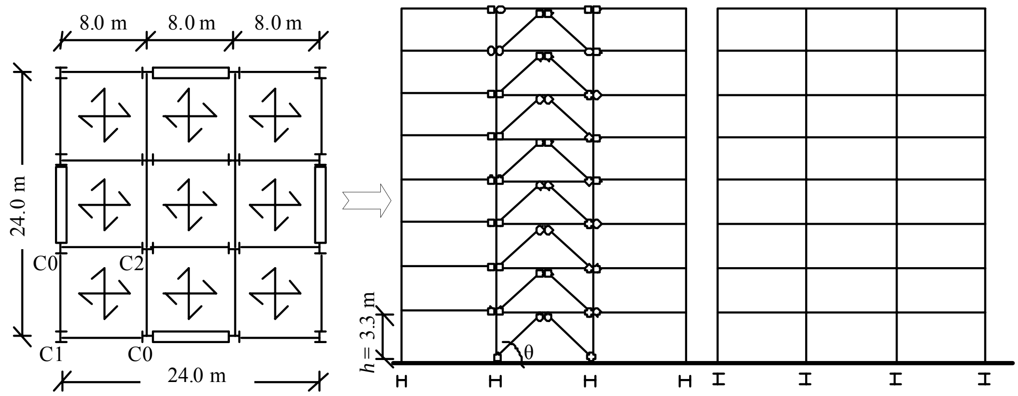

Figure 1. Layout of the dual structure

The braces, disposed in the chevron configuration, are located in the central span of the frames on the perimeter of the building. Columns are distinguished by labels C0, C1 and C2 (Fig. 1): central columns of MRFs are labeled as C2-type, central and outermost columns of braced frames are labeled as C0 and C1-type, respectively. C0-type columns are oriented in such a way that the lateral stiffness of these elements in the plane of the braced frame is dependent on the maximum value of the moment of inertia of the column cross-section. Columns of C1 and C2-type, instead, are oriented so as to provide the maximum lateral stiffness in the Y and X-directions, respectively. Pinned connections are considered between C0-type columns and beams belonging to the plane of the braced frame and at the base of C0-type columns in the plane of the braced frame. All other connections are assumed to be rigid and full strength.

Braces consist of square hollow cross-sections. European large flange sections HEB are used for beams belonging to the CBFs while IPE sections are used for beams belonging to the MRFs. With the exception of a single case columns are built of European large flange sections (HEB or HEM). The aforementioned exception refers to the columns C0 at the lower storeys of the twelve-storey building designed by the LFMA where two IPE sections have been welded to the web of a HEM section to obtain similar moments of inertia along the principal axes of the cross-section. Steel grade S235 is adopted for braces and for beams of MRFs. All the remaining members are built of steel grade S235, S275 or S355.

3.1. Overstrength factor of the designed concentrically braced frames

To verify the fulfilment of the homogeneity strength condition, the normalized overstrength factor O_s is first calculated at each storey as the ratio of the brace overstrength factor at the storey under examination Ω to the minimum value of the brace overstrength factor in the building Ω_{min}^{CBF} . The heightwise distribution of the normalized overstrength factor is shown in Figure 2a with reference to all the structures considered. In regard to this figure, dashes identify the minimum and maximum values of the normalised overstrength factor while circles, triangles and squares pinpoint the mean values of the normalised overstrength factor in the building.

The homogeneity strength condition is satisfied in all the cases under examination excluding the 12-storey building designed by the MRSA. The distribution of the normalized overstrength factors of this building has however been deemed to be acceptable because of the reasoning below. The normalized overstrength factors of the 12-storey building designed by the MRSA are close to 1 at all the storeys but the top one where it is equal to 1.28. In this situation, satisfying the upper limit of 1.25 would have required selecting larger cross-sections for the diagonals of the building. In addition, as remarked in other investigations (Elghazouli, 2010; Bosco and Rossi, 2009), the simultaneous presence of CBFs and MRFs leads to damage distribution capacity factors which are generally higher than those of the CBFs and thus to structures which are less sensitive to the scattering of the overstrength factor.

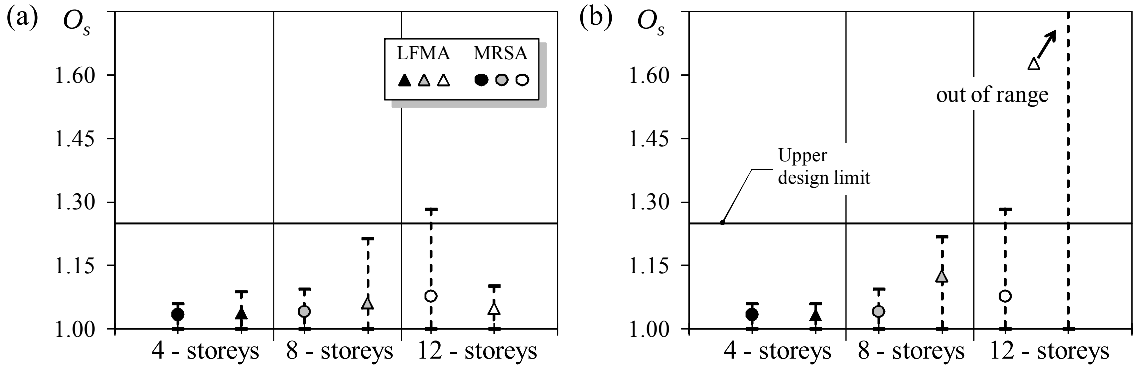

Figure 2. Distribution of the normalized overstrength factor evaluated by:
(a) the adopted design method of analysis, (b) modal response spectrum analysis

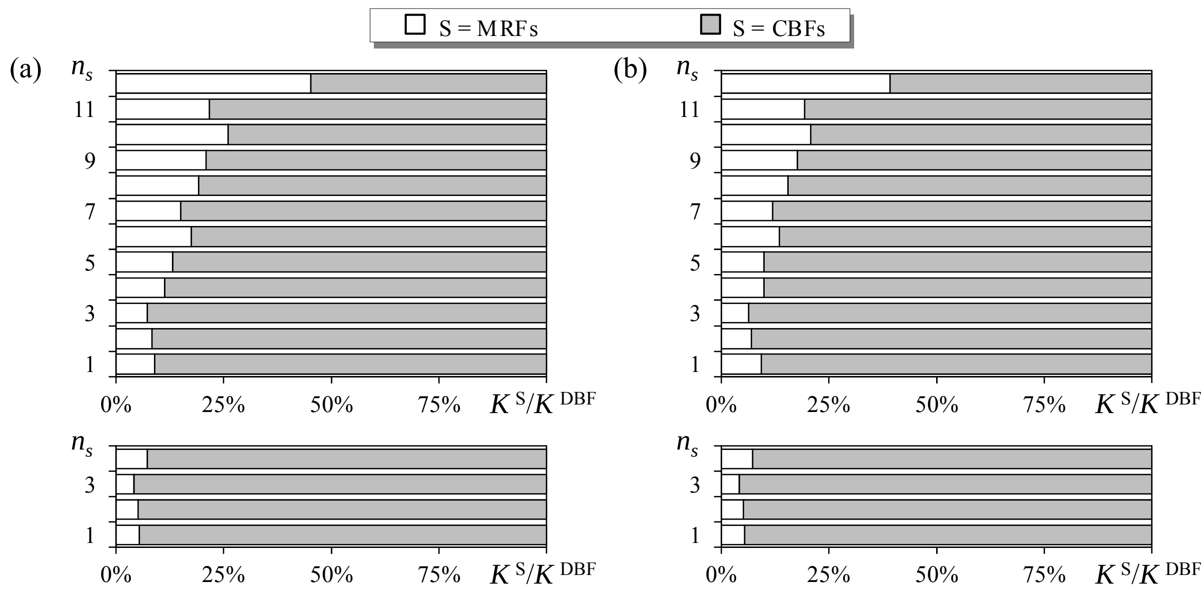

Figure 3. Contribution provided by the MRFs and CBFs to the stiffness of the dual system designed by: (a) modal response spectrum analysis, (b) lateral force method of analysis

Finally, it should be noted that when the lateral force method of analysis is used, the design storey shear is underestimated at the upper storeys. Thus the fulfilment of the homogeneity strength condition in high rise systems designed by the LFMA does not guarantee the simultaneous buckling of braces of all storeys. The validity of this statement is evident in Figure 2b where the normalized overstrength factors O_s of the buildings designed by the LFMA are calculated again on the basis of the internal axial forces N_{Ed} obtained by the MRSA. While the new maximum and mean values given for the 8-storey frame are only slightly higher than those in Figure 2a, the values obtained for the 12-storey frame are much higher than those calculated by LFMA. In addition, in this case, the new maximum value (2.51) and the mean value (2.10) are even significantly higher than the upper limit stipulated in the Eurocode 8, i.e. 1.25. It is to note that the fundamental period of vibration T_1 of the abovementioned 12-storey frame is equal to 1.60 s and thus no restrictions are applied by Eurocode 8 to the use of the LFMA.

3.2. Storey stiffness of the dual system

The heightwise distributions of the horizontal displacements experienced by CBFs and MRFs are generally very different. As a consequence, the contribution of the MRFs (K^{MRF}) to the lateral storey stiffness of the dual system (K^{DBF}) is usually non uniform in elevation. As an example, the heightwise distribution of the percentage ratio of the lateral stiffness provided by either of the two sub-structures to the stiffness of the dual structure is shown in Figure 3 with reference to the 4- and 12-storey systems. Note that the lateral forces considered for the calculation of the lateral storey stiffness are selected to be proportional to the shape of the first mode of vibration. As is evident, the ratio $K^{\mathrm{MRF}}/K^{\mathrm{DBF}}$ is about 0.10 at the lower storeys of the 12-storey buildings and increases with the storey level up to 0.45; the value 0.10 is, instead, practically constant in the 4-storey structures.

The ratio of the lateral stiffness of the MRFs over that of the entire dual system is considered by some researchers as a parameter able to synthetically describe the response of the structure in its first mode of vibration. This parameter is calculated here as the manner of Whittaker et al. (1988)

$$K^{MRF} / K^{DBF} = \left(T_1^{DBF} / T_1^{MRF}\right)^2 \tag{3.1}$$

where T_1^{MRF} and T_1^{DBF} are the fundamental periods of vibration of the MRFs and dual structure. Independently of the adopted design method of analysis, the ratio K^{MRF}/K^{DBF} is equal to 0.05 for the 4-storey structures and 0.07 for the 8-storey structures; the same parameter is equal to 0.13 and 0.11 for the 12-storey buildings designed by the MRSA and LFMA, respectively. In any case the ratio is much lower than the value 0.3÷0.5 proposed by Whittaker et al. for the design of dual braced systems.

3.3. Drift sensitivity coefficient θ

The drift sensitivity coefficient is calculated by Equation 2.5. Note that in the buildings where the design internal forces are determined by the MRSA the storey shear V_{Ed} and the storey drift Δu are obtained by combining the modal contributions by means of the SRSS rule. Further, in all the buildings the design storey drifts are calculated with reference to the effective value of q, i.e. the design behavior factor is reduced by the minimum overstrength factor Ω_{min}^{CBF} . As is evident in Table 3.1., the values of the parameter θ are always lower than 0.1. Therefore, P-Δ effects have been ignored in the phase of design.

Table 3.1. Maximum values of the drift sensitivity coefficient

	designed by MRSA	designed by LFMA
12 – storey frames	0.08	0.07
8 – storey frames	0.03	0.04
4 – storey frames	0.02	0.02

3.4. Damage limitation requirement

The fulfillment of the damage limitation requirement has not been forced in the phase of design but has been checked on the designed structures. The abovementioned requirement is considered verified if the storey drifts caused by frequent seismic actions are lower than a reference value stipulated in the code as a function of the type of non-structural elements present in the building. The storey drifts Δu are obtained here by multiplying the design storey drifts by 0.5 q. Reference limit values equal to 0.50% and 0.75% times the interstorey height are suggested for buildings with non-structural elements of brittle and ductile materials, respectively. A reference value of 1.00% times the interstorey height is suggested for buildings in which non-structural elements are fixed so as not to interfere with structural deformations. The heightwise distribution of the storey drifts normalized to the interstorey height ($\Delta u/h$) is plotted in Figure 4. As is evident, the maximum normalized storey drifts are lower than 0.50% in the 4-storey structures and lower than 0.75% in all the other cases.

4. NUMERICAL ANALYSIS

The seismic response of the dual systems is obtained by incremental nonlinear dynamic analysis. The single non-linear dynamic analysis is carried out by means of the OPENSEES program (Mazzoni et al., 2007). The peak ground acceleration a_g is scaled in step of 0.04g in order to estimate the peak ground acceleration corresponding to both first buckling of braces (or yielding of beams) and high damage in the structural elements. The latter reference level of damage is characterized by assigned axial deformations of braces, plastic rotations of ductile beams and columns belonging to the MRFs

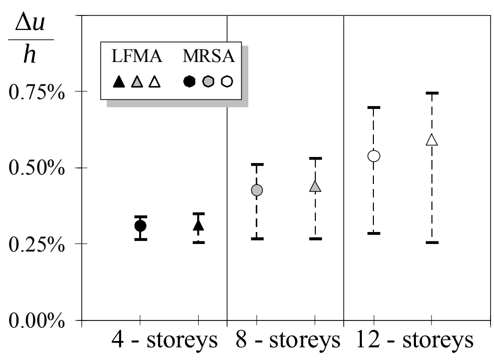

Figure 4. Distribution of the normalised storey drift

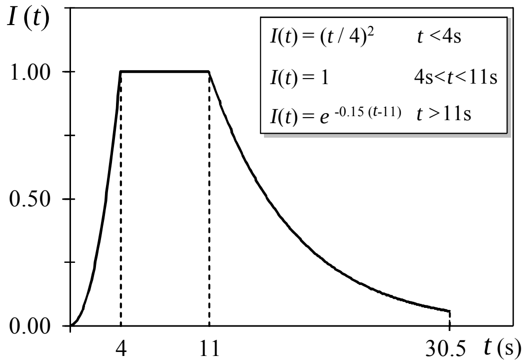

Figure 5. Compound envelope function

and assigned internal forces of brittle members (beams and columns of the CBFs, columns of the MRFs when subjected to high axial forces). The viscous damping forces are obtained through the formulation proposed by Rayleigh. In particular, 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 twice, i.e. either considering or neglecting the P-Δ effects.

4.1. Seismic input

The seismic input is constituted by ten accelerograms which are artificially generated and compatible with the elastic response spectrum proposed by the Eurocode 8 for soil C. As shown in Figure 5, these accelerograms are characterized by a total duration of 30.5 s and are enveloped by a "compound" function. It should be noted that the duration of the stationary part of the accelerograms is equal to 7.0 s and, therefore, lower than the minimum value suggested by the Eurocode 8, i.e. 10 s. The 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 (Amara, 2012).

4.2. Numerical model

The single brace is modeled by means of four "nonlinearBeamColumn" elements. The cross-section of the brace is divided into 20 fibers and the hysteretic behavior of steel is simulated by means of the model proposed by Menegotto-Pinto. An initial camber equal to 0.1% of the brace length is applied at brace mid-length. The corotational theory is used to simulate the moderate to large deformation effects on the inelastic behaviour of braces (Uriz et al., 2008). The plastic behavior of beams and columns of the MRFs is modeled by means of "beamWithHinges" elements. Beams and columns of the CBFs are expected to remain elastic and, therefore, are modeled by means of "elasticBeamColumn" elements.

4.3. Response parameters

Braces are verified by comparing the required ductility to the available ductility. The required ductility of the brace is defined as the sum of the maximum positive and negative axial deformations divided by the axial elongation of the brace at yielding. The available ductility of the brace μ_f is evaluated as 75% of the ductility at brace failure defined by Tremblay (2002)

$$\mu_f = 0.75\left(2.4 + 8.3 \ \overline{\lambda}\right) \tag{4.1}$$

where $\overline{\lambda}$ is the normalized slenderness of the brace.

The plastic rotation capacity at the ends of ductile beams and columns is expressed as a multiple of the chord rotation at yielding θ_y .In particular, as stipulated in the Eurocode 8-Part 3, the plastic rotation capacity is assumed equal to 6.0 θ_y if member sections are class 1 and equal to 2.0 θ_y if member sections are class 2. The chord rotation at yielding θ_y is calculated by means of the relation

$$\theta_y = \frac{M_{N,Rd}}{EI} \tag{4.2}$$

where $M_{N,Rd}$ is the design plastic moment resistance reduced due to the axial force, E is the Young modulus and I is the moment of inertia of the cross-section.

Beams and columns of the CBFs and columns with axial load equal or greater than 0.30 $N_{pl,Rd}$ are considered to be fragile. Therefore, no inelastic deformations or buckling phenomena are allowed in these members.

5. SEISMIC RESPONSE

The results of the incremental nonlinear dynamic analyses are shown in terms of peak ground accelerations corresponding to the achievement of specified reference states, namely first buckling of braces (Fig. 6a), deformation capacity of ductile members (Fig. 6b) and deformation capacity of ductile members or yielding (or buckling) of fragile members (Fig. 6c). In all the plots, circles identify mean values obtained for systems designed by the MRSA while triangles identify mean values obtained for systems designed by the LFMA. The range of variation of the results of the single accelerograms is also shown by means of arrows and the range of the peak ground accelerations corresponding to non-suitable performances is hatched. To highlight the effectiveness of the provisions stipulated in the Eurocode 8 with regard to the P-Δ effects the peak ground accelerations are calculated twice, either considering ($\alpha^{P-\Delta}$) or neglecting (α) P-Δ effects.

The mean value of the peak ground accelerations α_y corresponding to the first buckling of braces is generally lower than the value expected by design, i.e. α_{yd}=0.35g/2.5, and is not affected by P-Δ effects (Fig. 6a). Specifically, the mean value of α_y is equal to the value expected by design in the case of the 4-storey frames, while it is significantly lower than 0.14g when the LFMA is adopted to design the 12-storey frame.

The mean value of the peak ground accelerations α_u corresponding to the achievement of the available ductility of the ductile members is generally higher than the design value α_{ud}=0.35g: only the 12- and 8-storey frames designed by the LFMA are characterized by a mean peak ground acceleration (0.26g and 0.28, respectively) lower than the expected value (Fig. 6b). The behaviour factor, calculated as the

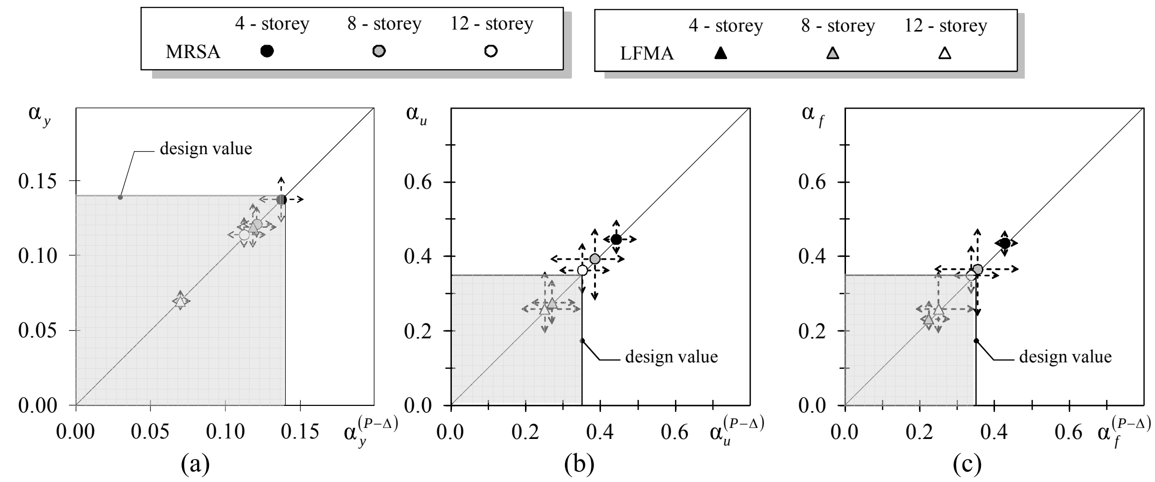

Figure 6. Peak ground acceleration corresponding to: (a) first buckling of braces, (b) deformative capacity of ductile members and (c) deformative capacity of ductile members or yielding (or buckling) of fragile members

ratio of α_u to α_{yd}, ranges from 1.85 to 3.18. The highest values of the behaviour factors are gained in the 4-storey frames and in the dual frames designed by the MRSA. P-Δ effects are generally negligible, a slight influence being found only in the 12-storey frame designed by the modal response spectrum analysis: in this case, the behavior factor decreases from 2.59 to 2.51 when P-Δ effects are included in the nonlinear dynamic analyses.

Finally, it is worth noting that the mean value of the peak ground accelerations α_f corresponding to both the attainment of the deformation capacity of ductile members and yielding or buckling of brittle members (Fig. 6c) is generally slightly lower than that corresponding only to the achievement of the available ductility of braces. Therefore, the simplified rules reported in the Eurocode 8 for the application of the capacity design principles do not penalize the seismic behavior of the buildings. The values of α_f are higher than the design value α_{ud}=0.35g in dual frames designed by the MRSA; because of this, the values of the behavior factor are close to the value adopted in design (2.5) in the 4-storey buildings and in the buildings designed by the MRSA, lower than 2.5 in all the other cases.

6. CONCLUSIONS

In this paper the reliability of the procedure proposed by the Eurocode 8 for the design of dual structures consisting of moment resisting frames and frames with braces in the chevron configuration is evaluated. To this end, six dual systems are designed according to the Eurocode 8 and their seismic response is evaluated by incremental nonlinear dynamic analysis.

The results of the analyses show that:

- the lateral force method of analysis for the evaluation of the design internal forces of braces should be avoided or allowed under more restrictive conditions;
- the dual systems examined are characterized by drift sensitivity coefficients lower than 0.1. As expected, P-Δ effects do not significantly affect their seismic response;
- the rules reported in the Eurocode 8 for the application of the capacity design principles are fairly effective although some yielding of columns of MRFs and buckling of columns of CBFs may occur prior to failure of braces;
- a value of the behaviour factor equal to 2.5 is generally conservative for systems in which the design internal forces are evaluated by modal response spectrum analysis;

REFERENCES

Amara, F. (2012) Strategie e misure di progetto per contrastare gli effetti PD in sistemi SDOF in zona sismica, tesi di Laurea. University of Catania (in Italian)

Bosco M., Rossi P.P. (2009) Seismic behaviour of eccentrically braced frames, Engineering Structures; **31**: 664-674.

Bosco M., Marino E.M., Rossi P.P. (2009) Influence of the brace overstrength on the seismic behavior of chevron braced frames, *Behaviour of Steel Structures in Seismic Areas STESSA 2009*, 297-302.

Elghazouli, A.Y. (2010). Assessment of European seismic procedures for steel framed structures. *Bulletin of Earthquake Engineering* **8**, 65-89.

Hines E.M. and Fahnestock L.A. (2010). Design Philosophy for Steel Structures in Moderate Seismic Regions, *9th US National and 10th Canadian Conference on Earthquake Engineering*.

Jain, A.K., Redwood, R.G. and Lu, F. (1993). Seismic response of concentrically braced dual steel. *Canadian Journal of Civil Engineering* **20**, 672-687.

Khatib I.F., Mahin S.A., Pister K.S. (1988) Seismic behaviour of concentrically braced steel frames. Earthquake Engineering Research Center, Report No. UCB/EERC-88/01. University of California

Tremblay R. (2002). Inelastic seismic response of steel bracing members, *Journal of Constr. Steel Research*, **58**: 665-701.

Uriz P., Filippou F.C. and Mahin S. A. (2008). Model for Cyclic Inelastic Buckling of Steel Braces, *Journal of Structural Engineering*, **134:4**, 619-628.

Whittaker A. S., Uang C., Bertero V.V. (1988). An experimental study of the behaviour of dual steel systems. Earthquake Engineering Research Center, Report No. UCB/EERC-88/14. University of California.