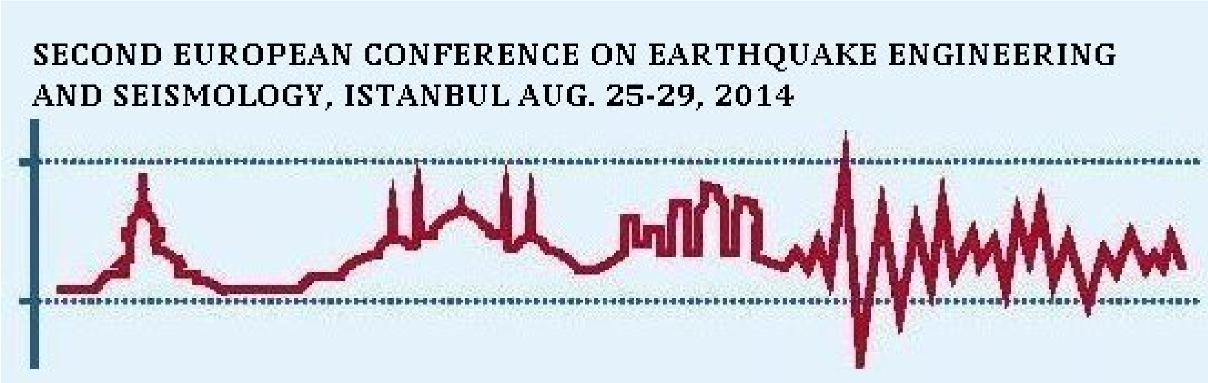

SEISMIC RESPONSE AND BEHAVIOUR FACTOR OF DUAL ECCENTRICALLY BRACED SYSTEMS DESIGNED BY EUROCODE 8

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ABSTRACT

The authors consider dual structures constituted by eccentrically braced frames and moment resisting frames and design these structures according to rules that are the natural extension of those proposed in Eurocode 8 for dual structures constituted by concentrically braced frames and moment resisting frames. The procedure considers that the dual eccentrically braced structures may be designed by means of a single value of the behaviour factor and that the lateral forces are distributed between the braced and moment resisting frames according to their lateral stiffness. In addition, a new design procedure is proposed, which slightly modifies that of Eurocode 8 in order to fill gaps and make the new procedure consistent with the intent of the code.

To evaluate the effectiveness of the rules proposed for the application of the capacity design principles and the adequacy of the behaviour factor, the seismic response of the designed systems is evaluated through incremental non-linear dynamic analysis. To ensure the general validity of the results, the designed buildings are considered to be founded on hard or soft soil and are characterised by different values of the link length and number of storeys.

INTRODUCTION

The seismic response of steel 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, i.e. they 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). For this reason, seismic codes required that the braced frames were subjected to the entire 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) and moment resisting frames are considered to be part of the primary lateral system. Consistently with this conception, Eurocode 8 (2005) requires 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, Eurocode 8 considers only dual structures constituted by moment resisting frames and concentrically braced frames. Further, the provisions and design criteria reported in other seismic codes cannot be directly transferred into the European code because of the different expected performance levels. These lacks

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limit the possibility of adopting these structural types or undermine the safety of the buildings because their design is entrusted exclusively to the experience of the structural designers.

The authors consider here dual structures constituted by eccentrically braced frames (EBFs) and moment resisting frames (MRFs) and design these structures according to rules that are the natural extension of those proposed in Eurocode 8 for other dual systems. The seismic response of these frames is analysed in order to evaluate the effectiveness of the rules proposed for the application of the capacity design principles and the adequacy of the behaviour factor considered.

DESIGN OF DUAL ECCENTRICALLY BRACED – MOMENT RESISTING FRAMES

The design procedure reported in Eurocode 8 for dual systems 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 two subsystems according to their lateral stiffness. The suggested value of the behaviour factor is equal or slightly higher than the lower value recommended for the two substructures. In the case of dual eccentrically braced – moment resisting frames, this minimum value is equal to $5\alpha_u/\alpha_1$, where α_u identifies the multiplier of the seismic design actions corresponding to the development of the overall structural instability and α_1 identifies that corresponding to the first yielding of members. However, recent studies on eccentrically braced frames (e.g. Bosco et al., 2014) have remarked that this value of the behaviour factor should be reduced. For this reason, a behaviour factor q_d equal to 5 is assumed.

The dual structures are designed so that yielding takes place in links, in beams of MRFs and at the base of the columns of the first storey of MRFs. For this reason, while these members are designed to resist the internal forces provided by the seismic design action, all the other elements are designed based on capacity design principles according to the provisions stipulated for the two substructures independently considered.

The design internal forces of the links may be obtained from either the lateral force method of analysis or modal response spectrum analysis. Link sections are selected so that the design shear force and bending moment (V_{Ed}, M_{Ed}) are not higher than the corresponding plastic resistances (V_p, M_p) calculated according to Eurocode 8. Links are defined as short, intermediate and long depending on the value of the mechanical length eV_p/M_p, e being the link length; specifically, if plastic hinges are expected at both ends of the link, links are short if the mechanical length is not greater than 1.6 and long if this ratio is not lower than 3. To achieve a global dissipative behaviour of the structure, Eurocode 8 recommends that the individual values of the link overstrength factor Ω_i^{EBF} do not exceed the minimum value Ω_{min}^{EBF} by more than 25% of this minimum value. The link overstrength factor Ω_i^{EBF} at the ith storey of the building is defined by means of the following relations

$$\Omega_i^{EBF} = \frac{1.5\,V_p}{V_{Ed}} \qquad \text{for short links} \tag{1}$$

$$\Omega_i^{EBF} = \frac{1.5\,M_p}{M_{Ed}} \qquad \text{for intermediate and long links} \tag{2}$$

Braces and columns of the braced frames are verified in compression by considering the most unfavourable combination of axial force and bending moment. Their section and steel grade are selected so that the following relation is verified

$$N_{Rd}\left(M_{Ed}, V_{Ed}\right) \geq N_{Ed,G} + 1.1\gamma_{ov}\Omega_{min}^{EBF} N_{Ed,E} \tag{3}$$

where $N_{Ed,G}$ is the compression force due to the non-seismic actions included in the seismic design situation; $N_{Ed,E}$ is the compression force due to the seismic design action; γ_{ov} is the material overstrength factor and N_{Rd} is the axial design resistance in accordance with Eurocode 3, taking into account the interaction with the bending moment M_{Ed} and the shear force V_{Ed}. These internal actions are calculated as the sum of the contributions of gravity loads and seismic forces to the design internal forces in the seismic design situation, i.e. $M_{Ed}= M_{Ed,G} + M_{Ed,E}$ and $V_{Ed}= V_{Ed,G} + V_{Ed,E}$.

Beams of MRFs are designed so that the design bending moment M_{Ed} at the ends of these members is not greater than the plastic bending moment $M_{pl,Rd}$. The selected sections are such that the design shear and axial forces do not decrease the full plastic moment and the rotation capacity at the plastic hinge.

The design internal actions of the columns of the MRFs are derived by the following relations

$$N_{Ed} = N_{Ed,G} + 1.1\gamma_{ov}\Omega_{min}^{MRF} N_{Ed,E} \tag{4}$$

$$M_{Ed} = M_{Ed,G} + 1.1\gamma_{ov}\Omega_{min}^{MRF} M_{Ed,E} \tag{5}$$

$$V_{Ed} = V_{Ed,G} + 1.1\gamma_{ov}\Omega_{min}^{MRF} V_{Ed,E} \tag{6}$$

where Ω_{min}^{MRF} is the minimum of the ratios $M_{plRd,i}/M_{Ed,i}$ calculated for all the beams of the MRFs in which dissipative zones are located. The column cross-sections are selected so that the design bending moment is lower than the flexural strength $M_{N,Rd}$ reduced by the axial force and 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).

Note that the design internal forces due to the seismic actions are amplified because of P-Δ effects if the interstorey drift sensitivity coefficient θ, calculated according to the following equation, is larger than 0.1 (Eurocode 8, 2005) and lower than 0.2.

$$\theta = \frac{P_{tot} d_r}{V_{tot} h} \tag{7}$$

In Equation (7) P_{tot} is the total gravity load at and above the storey considered in the seismic design situation, V_{tot} is the total seismic storey shear, h is the interstorey height and d_r is the design interstorey displacement at the storey under consideration. In Eurocode 8 this latter parameter is suggested to be calculated as a function of the design behaviour factor q_d.

DESIGNED BUILDINGS

The design procedure described in the previous section is applied to 2-, 4-, 8- and 12-storey buildings founded on hard to soft soils (soil types A, C and D 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 structural scheme is constituted by the intersection of two sets of four three-bay plane frames disposed along two orthogonal directions. Two eccentrically braced frames are located on the perimeter of the building along each of the two orthogonal directions. The eccentric braces are disposed in the central span in the split K geometric configuration. The link length e is assumed to range from 0.1 to 0.4 times the length L of the braced span. This range of values is considered to evaluate the seismic response of buildings in which the inelastic response of links is governed by either shear (short link) or flexure (long links), or by both shear and flexure (intermediate links). To nullify the interaction between deck and links, two beam members are introduced at each level of the EBFs instead of the traditional single section (Bosco et al., 2014). While the first sustains the vertical loads transmitted by the deck, the second resists the horizontal actions and constitutes the link itself. Columns are continuous in elevation and oriented as shown in Figure 1. Consequently, the stiffness provided by the moment resisting frames is greater along the x-direction because the columns labelled as C2 in Figure 1 are generally characterised by greater cross sections. Connections between columns of the braced frames and beams or foundation are pinned. All the other connections, including end connections of braces,

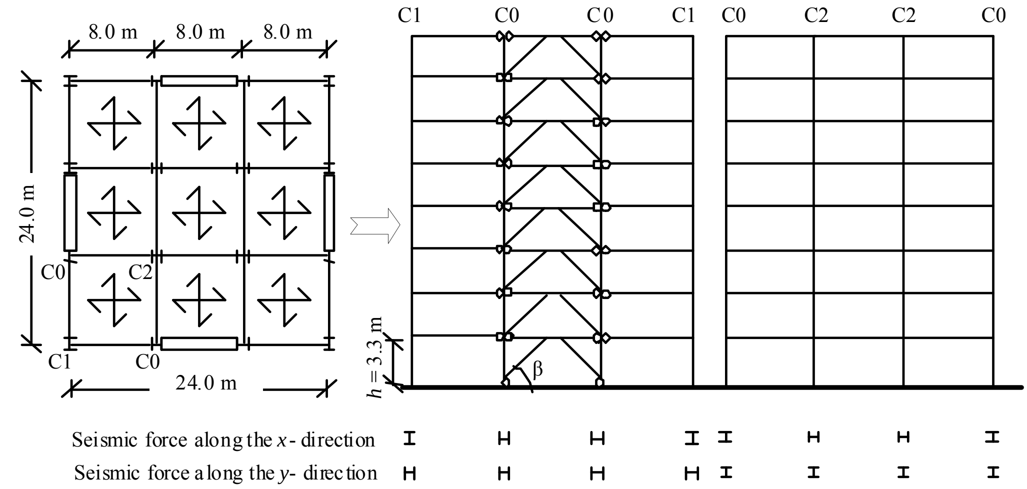

Figure 1. Layout of the dual structure

are rigid and full strength. The geometric and mass properties of the buildings are equal at all storeys. Vertical dead and live loads in the non-seismic design situation are equal to 9.2 kN/m^2. In the seismic design situation, the seismic action is calculated on the basis of masses corresponding to a mean value of the gravity loads equal to 5.0 kN/m^2. The internal forces due to the design seismic action 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.

In order to report synthetically the main properties of the designed structures, the ultimate available plastic rotation angle φ_u of the link of each storey is calculated first according to the relations provided by Eurocode 8, i.e.

$$\varphi_\mathrm{u} = \begin{cases} 0.08\ \mathrm{rad} & eV_\mathrm{p}/M_\mathrm{p} \le 1.6 \\ 0.02\ \mathrm{rad} & eV_\mathrm{p}/M_\mathrm{p} \ge 3.0 \\ \text{linear interpolation between the values above for } 1.6 < eV_\mathrm{p}/M_\mathrm{p} < 3.0 \end{cases} \tag{8}$$

Second, the mean plastic rotation capacity $\varphi_\mathrm{u}^\mathrm{m}$ of the links of the whole structure is calculated. This latter parameter is equal to 0.08 rad if all the links are short and equal to 0.02 rad if all the links are long; intermediate values are representative of systems with intermediate length links in at least one storey. Finally, the minimum, mean and maximum values of the normalised overstrength factors (Fig. 2) of each system are reported as a function of $\varphi_\mathrm{u}^\mathrm{m}$. Dashes identify the minimum and maximum values of the link normalised overstrength while triangles, rhombuses and circles pinpoint the mean value of the same parameter in the links of systems founded on different soil types. The maximum normalised overstrength factor is always lower than 1.25 for the EBFs arranged along the *y*-direction. Only sporadically, the maximum normalised overstrength factor is greater than 1.25 for high storey systems with long links arranged along the *x*-direction. Indeed, in these few cases the stiffness provided at the top storeys by the moment resisting frame is not negligible also in the elastic range of behaviour, thus the seismic forces at the upper storey are mainly sustained by the moment resisting frames. Further, the link cross section has not been reduced because the HEB 120 section has been selected as the minimum link cross section.

As remarked in Bosco et al. (2014), the expression provided in Eurocode 8 to evaluate the link overstrength factor Ω^EBF is not perfectly consistent to that suggested in the past by Popov et al. (1992). Specifically some differences may be found when at some storeys the link mechanical length is close to 1.6 because, as also reported by other researchers (Mazzolani et al., 2009), the link overstrength factor defined in Eurocode 8 is discontinuous at this specific value of the mechanical link length.

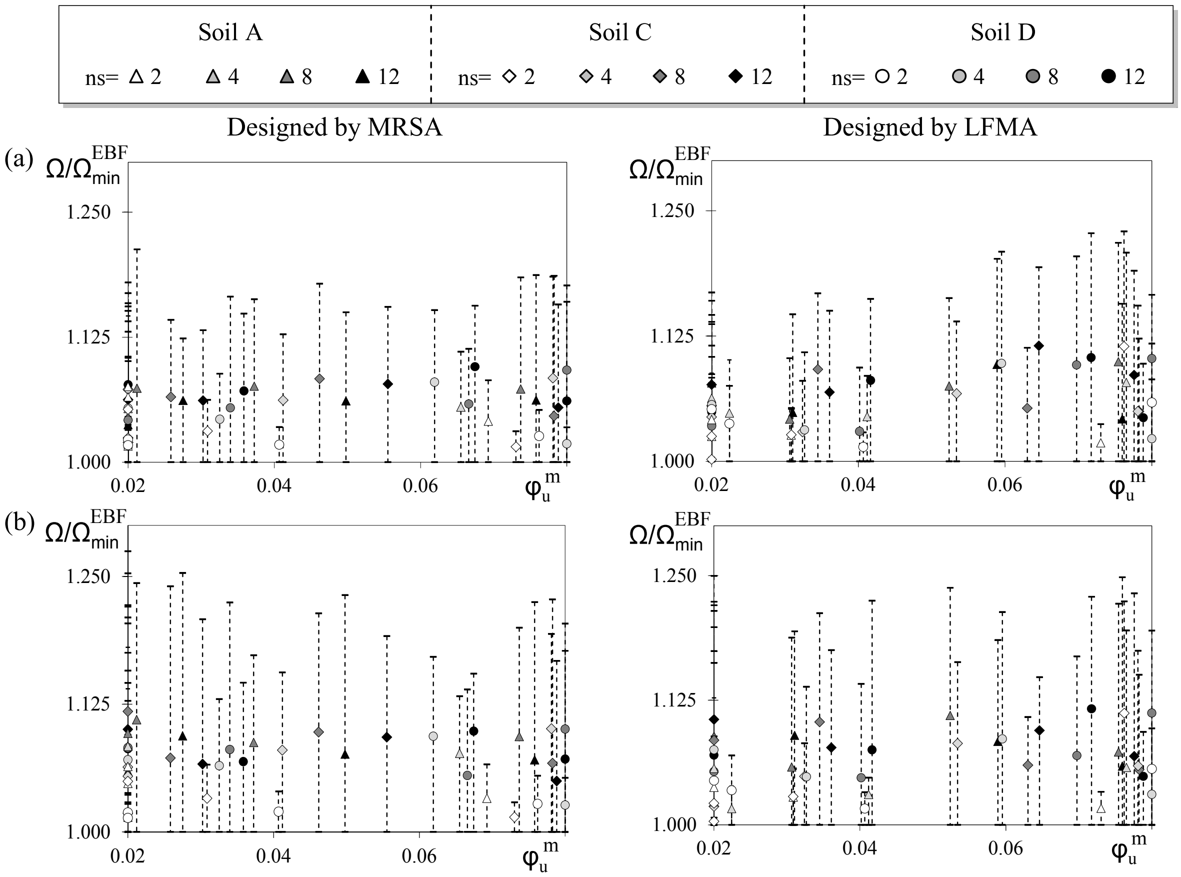

Figure 2. Maximum normalised overstrenght factors calculated according to Eurocode 8 for systems analysed along the (a) *x*-direction, (b) *y*-direction

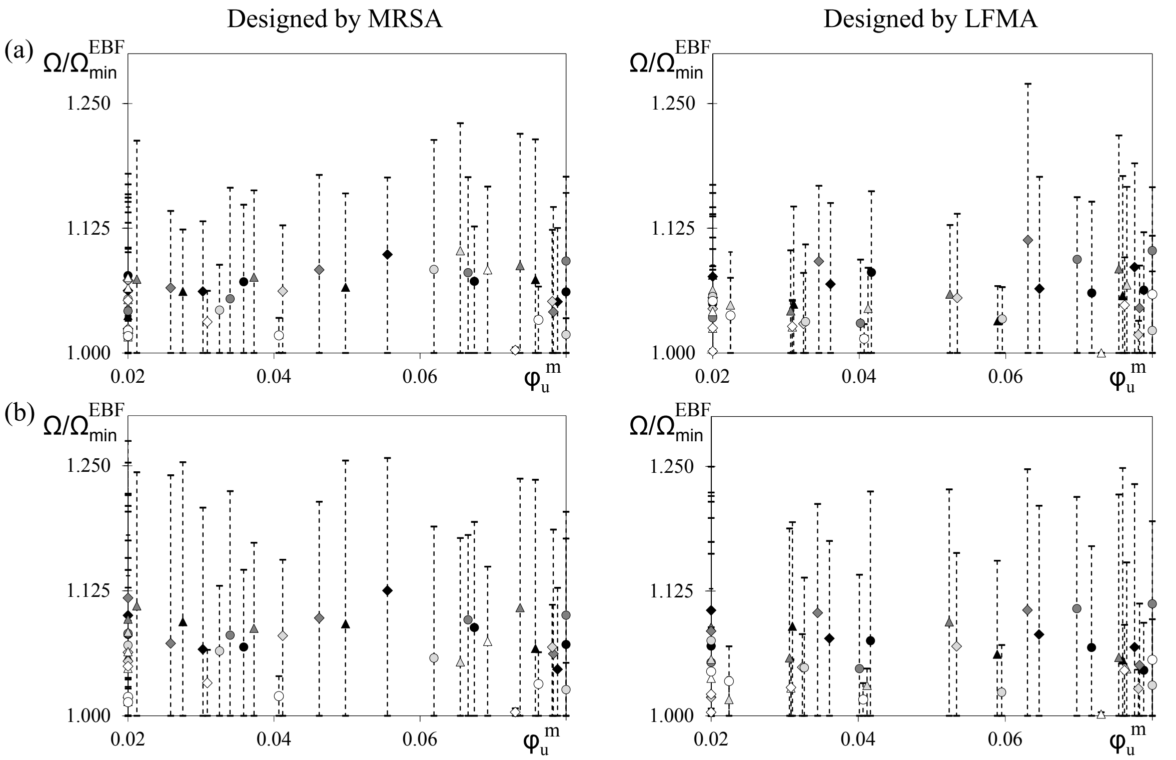

Figure 3. Maximum normalised overstrenght factors calculated according to Popov et al. (1992) for systems analysed along the (a) *x*-direction, (b) *y*-direction

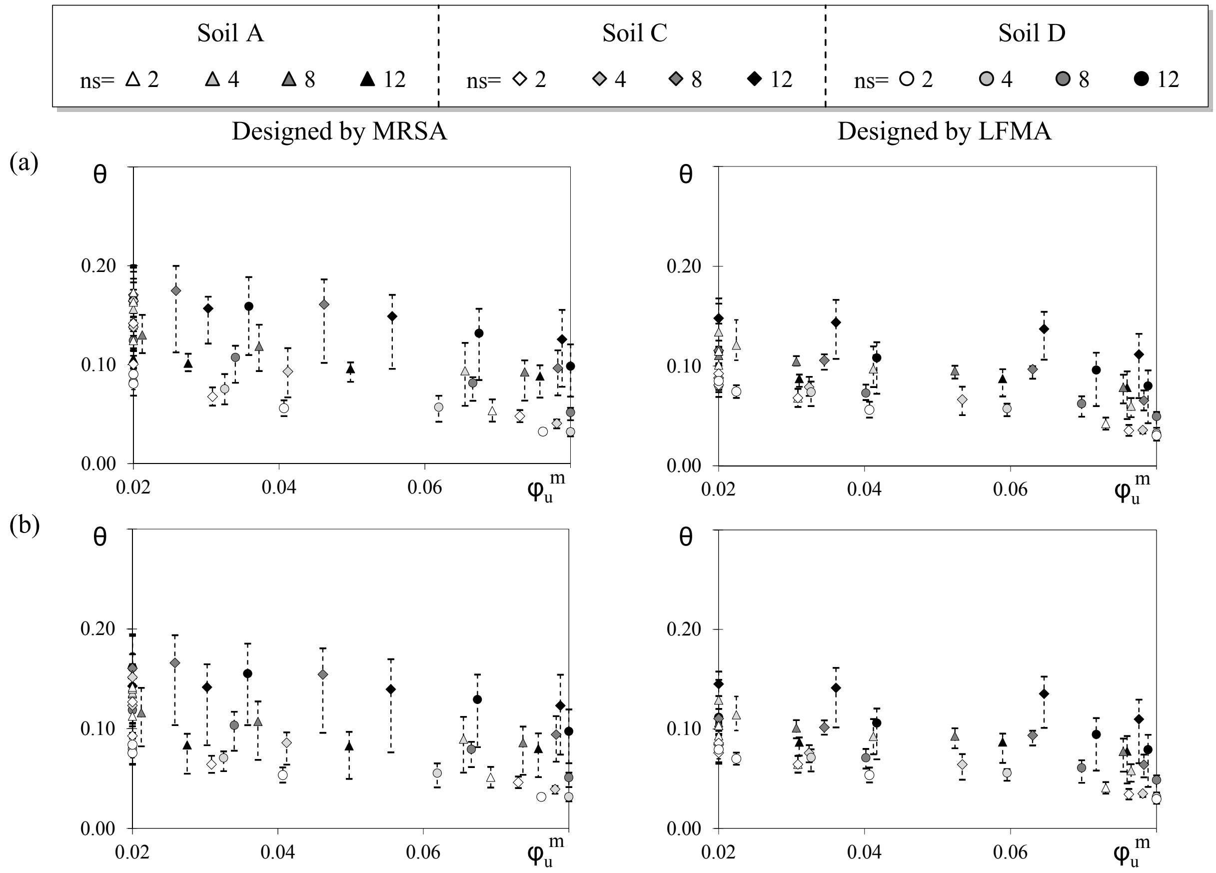

Figure 4. Interstorey drift sensitivity coefficients for systems analysed along the (a) *x*-direction, (b) *y*-direction

To show these differences, the normalised overstrength factors calculated according to the relation provided by Popov et al. (1992) are plotted in Figure 3. This figure shows that in some systems, i.e. those characterised by φ_u^m in the range 0.05 - 0.08, the maximum normalised overstrength factors may increase over 1.25 if the relation proposed by Popov is adopted.

Similarly, the maximum, minimum and mean values of the interstorey sensitivity drift coefficient are reported in Figure 4. The maximum value of the interstorey drift sensitivity coefficient θ along the height of the building generally increases as the mechanical length of the link increases (i.e. as the value of φ_u^m decreases). However, θ is always lower than 0.2.

NUMERICAL ANALYSES

The seismic response of the dual systems is obtained by incremental non-linear dynamic analysis. The single non-linear dynamic analysis is carried out on two-dimensional models by means of the OPENSEES program (Mazzoni et al., 2007). 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 the first and third modes of vibration of the 4-, 8- and 12-storey structures, and for the first and second modes of the 2-storey structures. For each structure, the analysis is performed including *P*-Δ effects and considering the seismic force action along either *x*- or *y*-direction.

Links are modelled by means of three elements connected in series (Bosco et al., 2014). The central element has the same length and moment of inertia of the link and simulates the flexural behaviour of the link (the shear stiffness of this element is infinite). The two ending elements are zero length and connect the beam segments outside the link to the flexural element of the link. The nodes of these ending elements are allowed to have only relative vertical displacements. The stiffness of the translational spring that allows this relative movement is defined so as to simulate the shear deformability of half a link. The post-elastic behaviour of the shear and flexural elements of the link is elasto-plastic with kinematic strain hardening. Braces, columns of the braced frames and beam segments outside links are modelled by means of elastic elements because they are non-dissipative members; beams and columns belonging to the moment resisting frames are modelled as beam with plastic hinges elements. Within the plastic hinge length (equal to the height of the cross section), the cross section is subdivided in fibres. The Menegotto-Pinto model is used to simulate the steel material.

The seismic input is constituted by three sets of ten accelerograms that are artificially generated and compatible with the elastic response spectrum proposed by the Eurocode 8 for soil A, C or D, respectively. These accelerograms are characterised by a total duration of 30.5 s and are enveloped by a “compound” function. The duration of the stationary part of the accelerograms is set equal to 7.0 s on the basis of 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 et al., 2014).

The peak ground acceleration a_g is scaled in step of 0.04g in order to estimate the peak ground acceleration corresponding to high damage in the structural elements (a_{gu}). The latter reference level of damage is characterised by assigned plastic rotations of links, beams and ductile columns belonging to the MRFs and assigned internal forces of non-dissipative members (beams and columns of the EBFs, columns of the MRFs when subjected to high axial forces).

DEFINITION OF THE STRENGTH OR DUCTILITY CAPACYTY OF MEMBERS

Links, beams of the MRFs and columns of the MRFs characterised by axial forces lower than 0.3 times the axial plastic resistance are ductile members. For link members the ductility capacity is given by Equation (8).

As reported in Eurocode 8 – Part 3, the inelastic deformation capacity for beams and columns in flexure is expressed in terms of the plastic rotation at the end of the member, as a multiple of the chord rotation at yielding θ_y

$$\theta_u = \begin{cases} 6\,\theta_y & \text{section class 1} \\ 2\,\theta_y & \text{section class 2} \end{cases} \tag{9}$$

θ_y is calculated as a function of the plastic flexural resistance reduced because of the axial force $M_{Rd,N}$, the shear span L_V, the moment of inertia of the cross section I and the modulus of elasticity E

$$\theta_y = \frac{M_{Rd,N} L_V}{3EI} \tag{10}$$

Thus, to assess the damage of these members, the maximum plastic rotations experienced for the seismic event scaled to the assigned value of a_g are normalised to the plastic rotation capacity above. The obtained ratios are indicated by the symbol R.

Braces, columns of EBFs and beam segments outside links are non-dissipative members. Their capacity is expressed in terms of the flexural strength $M_{N,Rd}$ reduced because of the axial force and in terms of the buckling resistance $N_{b,Rd\ (M)}$ reduced because of the bending moment. Thus, the safety levels of these members for both yielding and instability phenomena are evaluated by the ratios S and B between internal flexural or axial forces to the corresponding capacity.

Finally, columns of the MRFs that during the seismic event are subjected to axial forces N_{Ed} higher than $0.3\,N_{pl,Rd}$ are non-dissipative. For these columns, the ratio S cannot be calculated as reported above. In fact, these members are modelled as beam with plastic hinge elements and the maximum bending moment never reaches the flexural strength $M_{N,Rd}$ because of the gradual yielding of the single fibres of the hinge section. Thus, for these members, the safety level for yielding phenomenon S is calculated as the ratio of the maximum required curvature χ_{max} to the curvature at yielding χ_y, i.e as

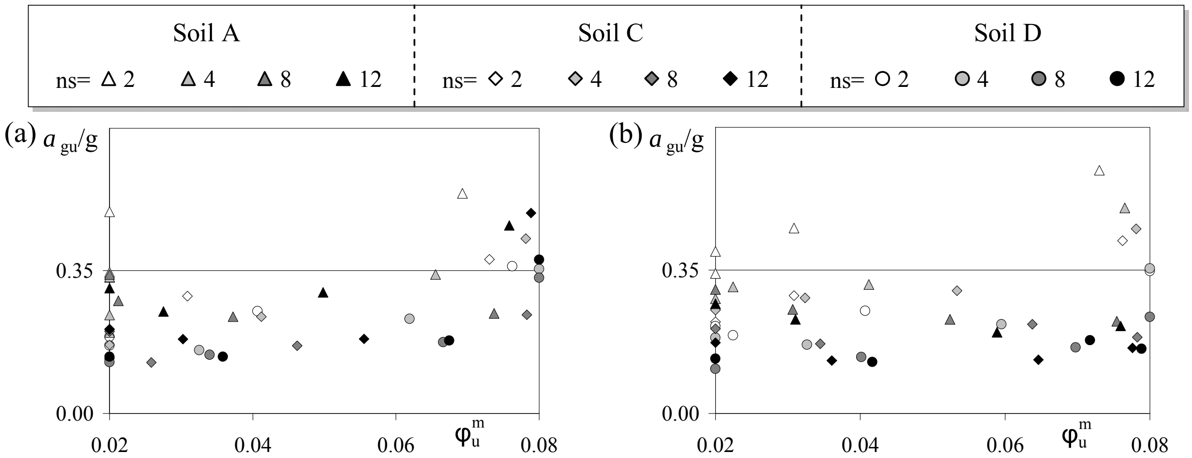

Figure 5. Peak ground acceleration corresponding to the first achievement of the ultimate limit state: dual frames designed by (a) MRSA; (b) LFMA

$$S = \frac{\chi_{\max}}{\chi_{y}} = \frac{\chi_{\max} EI}{M_{\mathrm{Rd,N}}} \tag{11}$$

ULTIMATE RESPONSE OF THE BUILDINGS

For each designed dual structure and for the single considered accelerogram, the peak ground acceleration a_{gu} corresponding to the first achievement of the ultimate limit state, i.e. to the first achievement of a unitary value of the ratios R, S or B defined in the previous section, is determined. Then, the value of a_{gu} is averaged over the values corresponding to the ten accelerograms and compared to the design peak ground acceleration (i.e. 0.35g). In Figure 5, the minimum peak ground acceleration a_{gu} obtained along the *x*- or *y*-direction is plotted as a function of the mean value φ_u^m of the ultimate plastic rotations of the links of the whole structure. The figure shows that, when the buildings are designed by the modal response spectrum analysis (Fig. 5a), values of a_{gu} larger than the design values are obtained only for some of the systems characterised by φ_u^m larger than 0.065. The range of dual systems with satisfactory behaviour is reduced further when the lateral force method of analysis is used (Fig. 5b). Indeed, in this case only 2- or 4-storey dual systems with φ_u^m larger than 0.065 reach the ultimate limit state for a_{gu} larger than 0.35g.

In order to investigate the reasons of this unsatisfactory behaviour, the number of accelerograms that lead to failure in links ($R = 1$), columns of MRFs (S or $B = 1$) or beam segment outside links (S or $B = 1$) is evaluated. In all the other members (braces and columns of EBFs and beams of MRFs) the seismic demand never exceeds the ultimate capacity. Figure 6 shows the results for some of the designed structures (4- and 12-storey dual systems founded on soil A and D designed by MRSA). This figure shows that the ultimate limit state is generally reached in links for dual systems in which the link length e is equal to $0.10\,L$. In the other systems, the ultimate limit state is often attained in the beam segments outside links and in non-dissipative columns of moment resisting frames earlier than in other members. Thus, the code provisions for the application of the capacity design principle do not always ensure the expected collapse mechanism.

The values of the ratios R, S or B corresponding to a seismic event with peak ground acceleration a_{gu} are averaged at each storey over the values corresponding to the ten accelerograms considered in the numerical analyses. The minimum, mean and maximum values of the parameter R obtained for links are reported in Figure 7 as function of φ_u^m for systems analysed along the *y*-direction. The figure shows that only in systems endowed with short links and designed by modal response spectrum analysis the mean value of R is high, which implies that the link damage is fairly uniform in elevation. In nearly all the other cases, the mean value of the link damage in the building is modest and identifies unsatisfying seismic performances. This behaviour can find two different reasons. In some cases (e.g. in structures with short links and designed by the lateral force method of

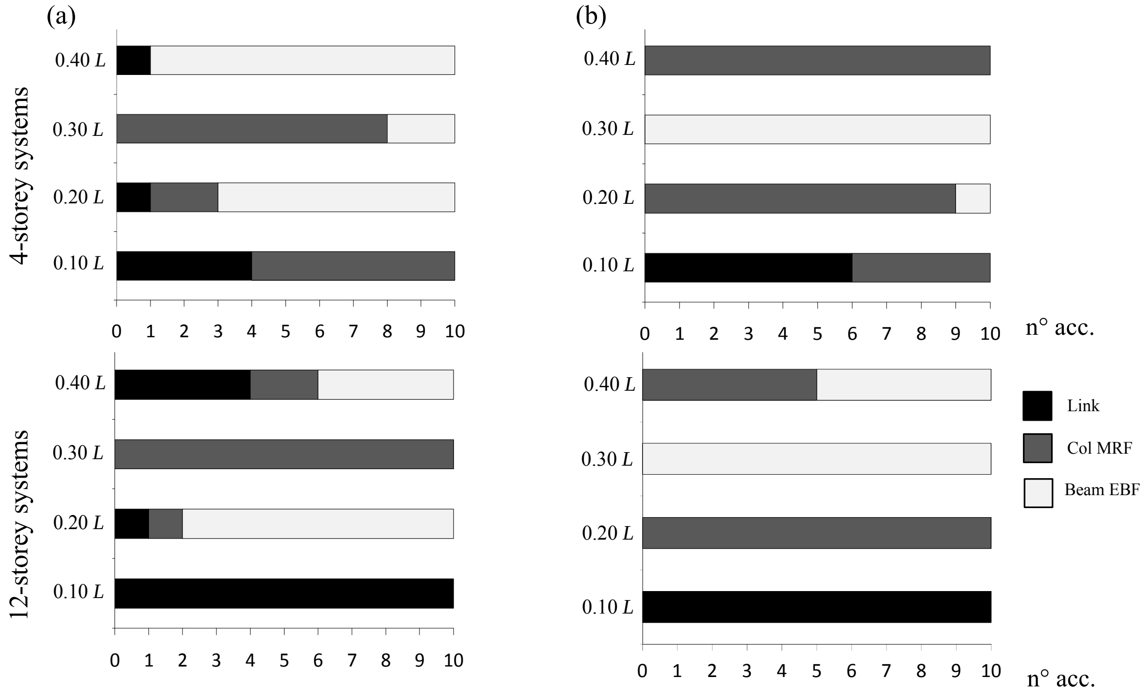

Figure 6. Number of accelerograms that lead to failure in links, beam segments outside links and columns of MRFs (seismic force along the *y*-direction): dual frames standing on (a) soil A; (b) soil D.

analysis), the ultimate limit state is achieved in links but the link damage is scattered in elevation. In other cases (e.g. in structures endowed with intermediate or long links) the ultimate limit state is achieved in non-dissipative members rather than in links and thus the link damage is everywhere much lower than unity.

Figure 8 shows that the plastic rotations in beams of the moment resisting frames are always low. In fact, the maximum value of R obtained for the beams of MRFs is always lower than 0.25.

PROPOSAL OF MODIFICATION OF THE DESIGN PROCEDURE

On the basis of the results described in the previous section, a new design procedure is proposed. This procedure is obtained modifying that of Eurocode 8. Some of the proposed modifications have recently been proposed for the design of eccentrically braced frames (Bosco et al., 2014), other modifications are related to the design of moment resisting frames (Elghazouli, 2009).

The first modification is related to the definition of the link overstrength factor. In particular, while the formulation of the overstrength factor reported in Eurocode 8 calculates how much the ultimate link strength exceeds the design internal force, the proposed overstrength factor Ω^{EBF} is referred to the first yielding of links. In fact, Ω^{EBF} is defined as the minimum between the following shear and flexural overstrength factors

$$\Omega_i^V = \frac{V_{p,i} - V_{Ed,G,i}}{V_{Ed,E,i}} \frac{\left(1 - \theta_i\, d_{e,i}/d_{r,i}\right)}{\left(1 - \theta_i\right)} \tag{12}$$

$$\Omega_i^M = \frac{M_{p,i} - M_{Ed,G,i}}{M_{Ed,E,i}} \frac{\left(1 - \theta_i\, d_{e,i}/d_{r,i}\right)}{\left(1 - \theta_i\right)} \tag{13}$$

where d_e is the interstorey displacement obtained from the structural analysis adopted in design and the ratio $\left(1 - \theta_i\, d_{e,i}/d_{r,i}\right)/\left(1 - \theta_i\right)$ is used to reduce the P-Δ effects to those corresponding to the first yielding of links. This correction is not required if the interstorey drift sensitivity coefficient θ is not higher than 0.10 because no additional strength is required by the code to counterbalance P-Δ effects

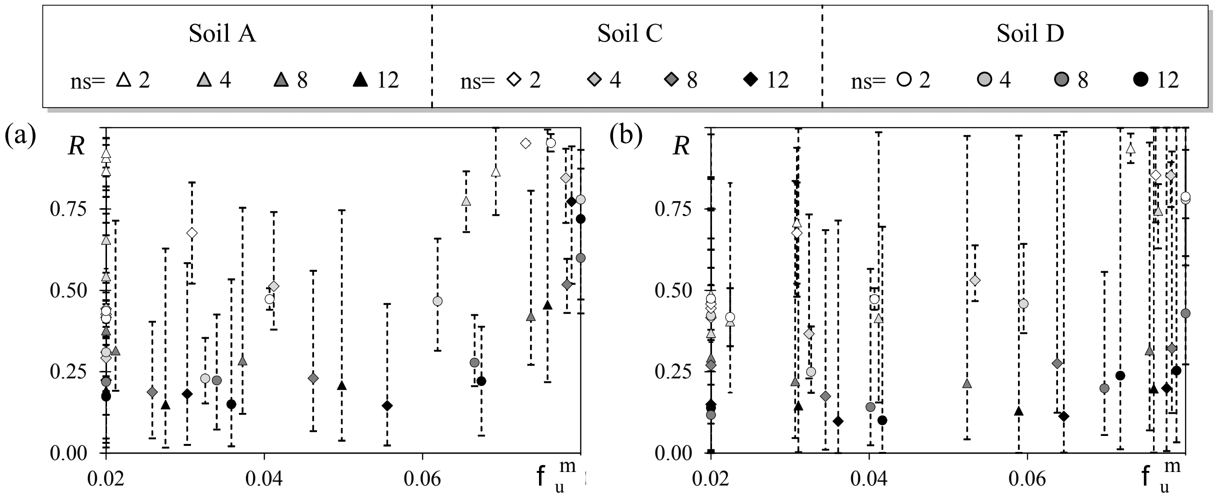

Figure 7. Ratio of the maximum plastic rotation to the available value for links: systems designed by (a) MRSA, (b) LFMA (*y*-direction)

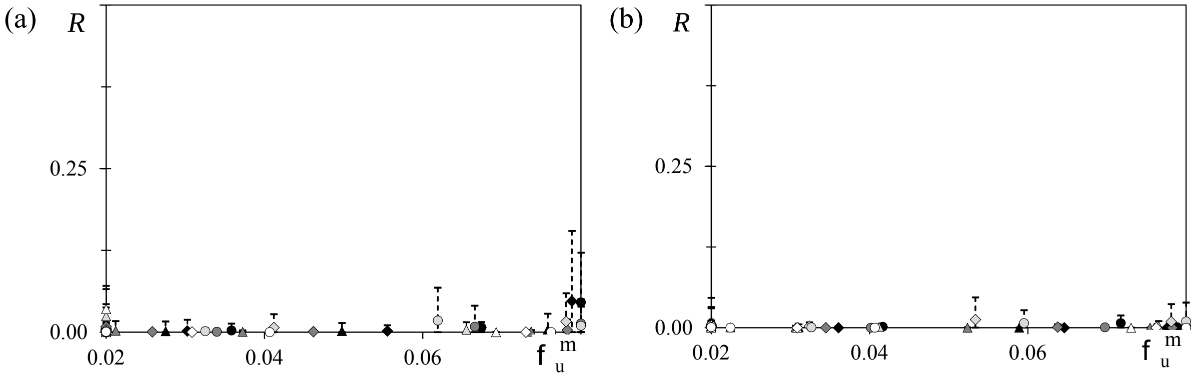

Figure 8. Ratio of the maximum plastic rotation to the available value for beams of the MRFs: systems designed by (a) MRSA, (b) LFMA (*y*-direction)

in this case. Note that the proposed definition of the overstrength factor is continuous with the mechanical link length and takes into account the internal forces caused by gravity loads.

Regarding the application of the capacity design principles, braces, columns of the EBFs and beam segments outside links are designed by means of the relations

$$N_{\mathrm{b,Rd}}\left(M_{\mathrm{Ed}}, V_{\mathrm{Ed}}\right) \geq N_{\mathrm{Ed}}\,;\; M_{\mathrm{N,Rd}}\left(N_{\mathrm{Ed}}, V_{\mathrm{Ed}}\right) \geq M_{\mathrm{Ed}} \tag{14}$$

where

$$N_{\mathrm{Ed}} = N_{\mathrm{Ed,G}} + 1.1\gamma_{\mathrm{ov}} 1.5 \cdot \Omega_{\min}^{\mathrm{EBF}} N_{\mathrm{Ed,E}} \tag{15}$$

$$M_{\mathrm{Ed}} = M_{\mathrm{Ed,G}} + 1.1\gamma_{\mathrm{ov}} 1.5 \cdot \Omega_{\min}^{\mathrm{EBF}} M_{\mathrm{Ed,E}} \tag{16}$$

$$V_{\mathrm{Ed}} = V_{\mathrm{Ed,G}} + 1.1\gamma_{\mathrm{ov}} 1.5 \cdot \Omega_{\min}^{\mathrm{EBF}} V_{\mathrm{Ed,E}} \tag{17}$$

In the equations above, the internal forces are calculated as a function of the ultimate internal forces of the links. In fact, the coefficient 1.5 appears in these equations because it is not included in the overstrength factor.

Finally, as suggested by Elghazouli (2009) with regard to moment resisting steel structures, the mathematical formulation of the overstrength factor $\Omega_{\min}^{\mathrm{MRF}}$ should take into account the effects of the gravity loads, i.e. it should be defined as the minimum of the ratios $(M_{\mathrm{plRd,i}} - M_{\mathrm{Ed,G,i}})/M_{\mathrm{Ed,E,i}}$ calculated for all the beams of the MRFs.

Table 1. Comparison of the seismic responses of the 8-storey dual systems (e = 0.30 L, soil A) designed by the two considered procedures

Design Procedure	T_1 (s)	φ_u^m (rad)	a_{gu}	N° acc Link failure	N° acc Col. MRF failure	N° acc Beam EBF failure
Eurocode 8	1.93	0.021	0.280 g	0	4	6
Proposed	1.64	0.023	0.485 g	10	0	0

VALIDATION OF THE PROPOSED PROCEDURE

The effectiveness of the proposed procedure is shown with reference to the 8-storey building with e/L equal to 0.30 and standing on soil type A. Table 1 shows that the dual frame designed by the proposed modification is stiffer than that designed by the procedure reported in Eurocode 8. In fact, the flexural rigidity of braces has been increased so as to reduce the bending moments in the beam segments outside links and verify the resistance requirements (see Eq. 14) in these members. The link sections adopted in the two compared systems are similar. In fact, the obtained values of φ_u^m are close each other. Table 1 also reports the comparison of the seismic responses of the two frames and shows that, when the proposed modifications are applied, the ultimate limit state is always reached in links and the peak ground acceleration a_{gu} is higher than 0.35 g.

Figure 9 shows the comparison between the two dual structures in terms of the hightwise distribution of the normalised plastic rotations of links. Specifically, the white circles identify the normalised plastic rotations experienced by the links on the occurrence of the ten accelerograms, each scaled to a peak ground acceleration equal to $a_{gu,i}$. At each floor, the black circle pinpoints the mean value of the normalised plastic rotations produced by the ten accelerograms and thus identifies the value of the abovementioned parameter R at that storey. In the dual frame designed according to Eurocode 8 the link damage is everywhere much lower than unity because the ultimate limit state is achieved in non-dissipative members. In the dual frame designed by the proposed approach, instead, the maximum normalised plastic link rotation caused by the single accelerogram is equal to unity. Further, the maximum normalised plastic rotations do not concentrate at the same storey level (i.e. the

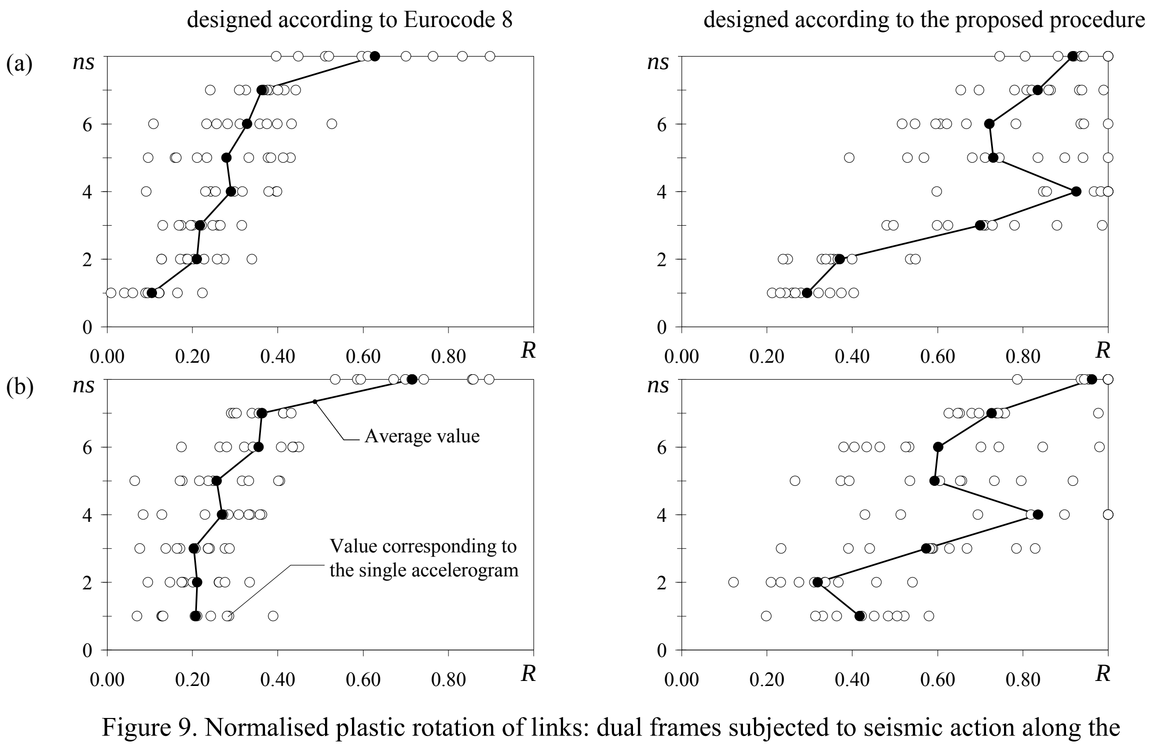

Figure 9. Normalised plastic rotation of links: dual frames subjected to seismic action along the (a) x- direction; (b) y-direction.

accelerograms do not cause the plastic rotation capacity of the links to be reached at the same storey of the building) and thus the maximum value of R is slightly lower than unity. This result is encouraging but it should be validated by means of the analysis of a wide set of dual systems.

CONCLUSIONS

The paper evaluates the effectiveness of the procedure suggested in Eurocode 8 for the design of dual structures obtained by coupling eccentrically braced frames and moment resisting frames. The incremental non-linear dynamic analyses performed on the designed dual structures show that a value of the behaviour factor equal to five ensures a satisfying ultimate performance only for structures endowed with short links and (i) designed by the modal response spectrum analysis or (ii) characterised by a low number of storeys and designed by lateral force method of analysis. In all the other cases, the peak ground acceleration corresponding to the achievement of the ultimate limit state is lower than 0.35g. These low values are chiefly caused by the limited effectiveness of the rules for the application of the capacity design principles.

For this reason, some modifications are proposed to the design procedure of Eurocode 8. The effectiveness of the proposed modifications is shown with reference to a single case study, i.e. an 8-storey systems endowed with intermediate link. The obtained results are encouraging but they should be validated by means of the analysis of a wide set of dual systems.

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