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Critical review of the EC8 design provisions for buildings with eccentric braces

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Abstract. The objections raised by researchers to the design provisions reported in Eurocode 8 make the efficient seismic performance of the eccentrically braced structures designed according to this code unlikely. Given the rationality and the number of the objections, this paper aims to summarize the criticism of researchers and report the opinion of the Authors. The objections raised to the design procedure of eccentrically braced structures regard aspects common to the design of steel structures and aspects specifically related to the design of eccentrically braced structures. The significance of these objections is also shown by means of exemplary cases.

Keywords: seismic areas; eccentrically braced frames; Eurocode 8; design provisions; capacity design

1. Introduction

Since the last decades of the twentieth century (Roeder and Popov 1978) the mechanical characteristics of the eccentrically braced systems have drawn the attention of researchers and civil engineers. The link beam, i.e., the element designated to dissipate energy on the occurrence of strong ground motions, has been extensively investigated from both experimental and numerical points of view. The laboratory activity has regarded single links as well as frame subassemblages. The seismic response of links has been simulated to comprehend the importance of their mechanical and geometric parameters while that of frame subassemblages to evaluate the reliability of the link connection to the adjacent members (Malley and Popov 1983, 1984, Hjelmstad and Popov 1984, Kasai and Popov 1986a, b, Engelhardt and Popov 1989a, Ricles and Popov 1989, Itani *et al*. 2003, McDaniel *et al*. 2003, Richards and Uang 2005, Chao *et al*. 2005, Okazaki *et al*. 2006a, b, Okazaki and Engelhardt 2007, Della Corte *et al*. 2007, Okazaki *et al*. 2009, Dusicka *et al*. 2010). Some tests have also been carried out on low-rise plane frames (Roeder and Popov 1978, Whittaker *et al*. 1987, 1988 Foutch 1989) to verify the anticipated seismic response and highlight possible gaps or inadequate provisions in the design process. Some other tests have been carried out on RC frames equipped with eccentric bracings (Mazzolani *et al*.

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2009a). Based on these theoretical and experimental results, design rules have been proposed for links, braces, columns and beam segments outside links (Engelhardt and Popov 1989b, Popov *et al*. 1992, Lu *et al*. 1997, Kasai and Han 1997). In compliance with the capacity design principles, the provisions intended for links have been adjusted to ensure a cyclic response which is dissipative and stable as that of other dissipative elements (Xie 2005). Instead, the provisions regarding braces, columns and beam segments outside links have been proposed in order to preserve the elastic response of these members until the collapse of the whole system has been reached.

Rules for the seismic design of eccentrically braced frames in European technical documents date back to the early 1990s and were reported in documents issued by the European Convention for Constructional Steelwork (ECCS 1991, 1994). Some years later, similar design provisions were introduced in Eurocode 8 - Part 1-3 (1995). Thereafter, numerous modifications have been made to the design rules of the eccentrically braced structures.

The evaluation of the effectiveness of the rules of Eurocode 8 for the design of eccentrically braced structures has been the object of only a few research studies (Mastandrea *et al.* 2001, Badalassi *et al.* 2013, Bosco *et al.* 2014). In these papers and in others as well, many objections have been raised to these design provisions (Rossi and Lombardo 2007, Elghazouli 2007, 2010, Mazzolani *et al.* 2009b). Given the rationality and the number of the objections, this paper aims to summarize the criticism of researchers and report the opinion of the Authors. The significance of the objections is also shown by means of exemplary cases. To clarify the objections, the design approach and the rules proposed in Eurocode 8 are first reported.

2. The expected seismic response of eccentrically braced structures

The European seismic code is based on a dual-level approach. Thus, the performance on which a direct verification is made refers to seismic actions characterized by two different intensities. The seismic action associated with the no-collapse requirement can cause damage in the links but cannot produce any inelastic phenomenon (yielding or buckling) in the other resisting members of the structure (braces, columns and beam segments outside links). This concept, reported in Eurocode 8 - Part 1 (2003), is expressed in more detail in Part 3 (2005), which is devoted to the seismic assessment of existing buildings.

The damage produced by the seismic action associated with the damage limitation requirement is deemed to be acceptable if the maximum interstorey displacement is smaller than an assigned limit value. This limit value is fixed on the basis of the type of connection between structural and non-structural elements and ranges from 0.005 to 0.010 times the interstorey height.

3. The design procedure reported in Eurocode 8

The design procedure proposed in Eurocode 8 for eccentrically braced systems applies to structures in which links are in either horizontal or vertical position (Fig. 1) and are able to dissipate energy by yielding in flexure and/or shear. This design procedure is founded on the force-based approach. Further, the design rules comply with the capacity design principles and are intended to ensure a homogeneous dissipative behaviour of the links.

The design internal forces of the links may be obtained from either the lateral force method of analysis or modal response spectrum analysis. The upper limit values of the behaviour factors depend on the expected ductility class of the structure and on the regularity of the system. If the eccentrically braced system is regular in plan and in elevation, the maximum recommended value of the behaviour factor of medium ductility class structures is equal to 4; the upper limit of the behaviour factor increases to 5 times α_u/α_1, when referring to high ductility class structures. The parameter α_u/α_1 (≥ 1) represents the overstrength of the whole system and can be obtained by a nonlinear static (pushover) analysis. The parameter α_1 is the value by which the horizontal seismic design action is multiplied to first reach the plastic resistance in any member of the structure; the parameter α_u is the value by which the horizontal seismic design action is multiplied to form plastic hinges in a number of sections sufficient for the development of overall structural instability. Even though the nonlinear static analysis indicates very high global overstrength factors, the value of α_u/α_1 to be considered in design cannot be higher than 1.6. If the abovementioned calculation of the parameter α_u/α_1 is not performed, Eurocode 8 allows designers to use a default value equal to 1.2 for eccentrically braced systems. The European code suggests reducing the behaviour factor by 20% if the building is non-regular in elevation. In addition, if buildings are not regular in plan, the value of the parameter α_u/α_1 should be assumed equal to the average of unity and the value considered for buildings that are regular in plan and in elevation.

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 (Fig. 1) and V_p and M_p the plastic shear and bending moment resistances of the link. If only one plastic hinge is expected in the link (e.g., see Fig. 1(b)) this classification also depends on the parameter α which is the ratio of the smaller bending moment at one end of the link in the seismic design situation, to the greater bending moment at the end where the plastic hinge would form, both moments being taken as absolute values. Eurocode 8 states that for I sections the length that divides short links from intermediate length links is equal to $0.8(1+\alpha)M_p/V_p$ and that the length which divides long links from intermediate length links is equal to $1.5(1+\alpha)M_p/V_p$. If plastic hinges are expected at both ends of the link, α is equal to 1 and, therefore, links are short if the mechanical length eV_p/M_p is not greater than 1.6 and long if the ratio eV_p/M_p is not lower than 3.

If the design value of the axial force of the link in the seismic design situation is not greater than 0.15 times the axial plastic resistance of the link, i.e., $N_{Ed}/N_{pl,Rd} \leq 0.15$, the following conditions must be satisfied at both ends of the link

$$V_{Ed} \leq V_p \tag{1}$$

$$M_{Ed} \leq M_p \tag{2}$$

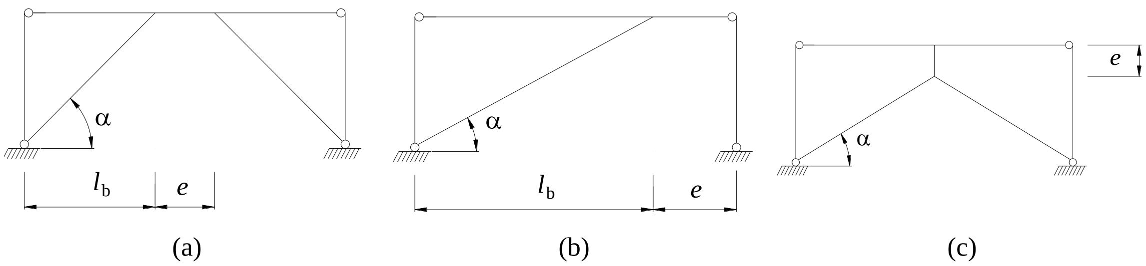


Fig. 1 Different geometric configurations of eccentrically bracings

where N_{Ed}, M_{Ed}, V_{Ed} are the design values of the axial force, bending moment and shear force of the link.

Eurocode 8 defines the link overstrength Ω_{i} which is evaluated for each link as

$$\Omega_{\mathrm{i}} = 1.5\, V_{\mathrm{p,i}} / V_{\mathrm{Ed,i}} \qquad \text{for short links} \tag{3}$$

$$\Omega_{\mathrm{i}} = 1.5\, M_{\mathrm{p,i}} / M_{\mathrm{Ed,i}} \qquad \text{for intermediate and long links} \tag{4}$$

and recommends that the individual values of the ratios $\boldsymbol{\Omega}_i$ do not exceed the minimum value $\boldsymbol{\Omega}_{\mathrm{min}}$ by more than 25% of this minimum value. This provision, which is common to other low redundant structural types (Zona *et al*. 2012, Bosco and Marino 2013, Marino 2014) aims at achieving a global dissipative behaviour of the structure and is of uttermost importance. Indeed, several studies (e.g., Popov *et al.* 1992, Bosco and Rossi 2009), have proved that systems with scattered values of the normalized link overstrength factors are prone to develop partial or storey collapse mechanisms.

The members not containing seismic links (columns and diagonals and also beam members if vertical links are used) are verified in compression considering the most unfavourable combination of the axial force and bending moments

$$N_{\mathrm{Ed}} \le N_{\mathrm{Rd}}\left(M_{\mathrm{Ed}}, V_{\mathrm{Ed}}\right) \tag{5}$$

where N_{Rd} is the axial design resistance of the member in accordance with Eurocode 3, taking into account the interaction with the bending moment M_{Ed} and the shear V_{Ed} considered at their design value in the seismic situation and

$$N_{\mathrm{Ed}} = N_{\mathrm{Ed,G}} + 1.1\gamma_{\mathrm{ov}}\, \Omega_{\mathrm{min}}\, N_{\mathrm{Ed,E}} \tag{6}$$

In Eq. (6), $N_{\mathrm{Ed,G}}$ and $N_{\mathrm{Ed,E}}$ are the axial forces due to the gravity and seismic actions in the seismic design situation. In regard to the calculation of M_{Ed} and V_{Ed}, the Authors note that Eurocode 8 does not specify any particular analytical expression. Neither does Eurocode 8 refer to other particular expressions of the bending moment and shear force, i.e., those used for column of moment resisting structures. Owing to this, these internal forces should be 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., $M_{\mathrm{Ed}}=M_{\mathrm{Ed,G}}+M_{\mathrm{Ed,E}}$ and $V_{\mathrm{Ed}}=V_{\mathrm{Ed,G}}+V_{\mathrm{Ed,E}}$, as already considered in reference (Bosco *et al.* 2014).

To counteract the P-Δ effects, Eurocode 8 amplifies the seismic action effects obtained by the design structural analysis. This amplification is stipulated on the basis of the values of an interstorey displacement sensitivity coefficient θ calculated as

$$\theta = \frac{P_{\mathrm{tot}}\, d_{\mathrm{r}}}{V_{\mathrm{tot}}\, h} \tag{7}$$

where P_{tot} is the total gravity load in the seismic design situation at and above the storey considered, d_{r} is the design interstorey displacement at the storey under consideration, V_{tot} is the total seismic storey shear and h is the interstorey height. According to Eurocode 8, the interstorey displacement d_{r} is calculated as the difference of the average lateral displacements d_{s} at the top and bottom of the storey under consideration; in turn, the lateral displacement d_{s} is the elastic design displacement d_{e} times the displacement behaviour factor q_{d}. The value of d_{s} does not need to be larger than the value derived from the elastic spectrum. No amplification of the seismic action effects is required if $\theta \leq 0.1$. If $0.1 < \theta \leq 0.2$, the second order effects may appropriately be taken into account by a simplified approach, i.e., by multiplying the relevant seismic action effects by a factor equal to $1/(1-\theta)$. If $\theta > 0.2$, the simplified approach is not applicable and a second order analysis has to be performed. The value of the parameter θ cannot be greater than 0.3.

4. Objections to the provisions of Eurocode 8

The objections raised to the design procedure of eccentrically braced structures regard both aspects related to the design of generic steel structures and aspects specific to the design of eccentrically braced structures. These objections are discussed in the following sections giving priority to those relative to the evaluation of the internal forces of the entire system. Particular attention is paid to the evaluation of the link overstrength factor because of the recognized importance of this factor for the response of eccentrically braced structures.

5. Behaviour factor

Eurocode 8 proposes different upper limits for the behaviour factor of high and medium ductility class buildings. The cross-sectional class of the link beams is the only parameter necessary for the evaluation of the ductility class of the abovementioned structures. According to Eurocode 3 (2003), the cross sections are classified to identify the extent to which the resistance and rotation capacity of cross sections is limited by its local buckling resistance. Specifically, class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance; class 2 cross-sections can develop their plastic moment resistance but have limited rotation capacity because of local buckling. The classification of a cross-section depends on the width to thickness ratio of the parts subject to compression and on the yield stress of the steel (Marino *et al.* 2005). Link beam sections of eccentrically braced systems are usually class 1 and, therefore, the behaviour factor corresponding to medium ductility class buildings is rarely used. Further, note that the limit reference behaviour factor suggested in Eurocode 8 for the design of eccentrically braced frames is independent of the mechanical link length, i.e., the behaviour factor is equal for short, intermediate and long links. This choice is in contrast with the results of some recent studies (e.g., Rossi and Lombardo 2007, Bosco and Rossi 2013a, b) where the behaviour factor is calculated a posteriori on the basis of nonlinear dynamic analyses of a large set of systems with short to long links.

The analytical relationship reported in Eurocode 8 for the calculation of the upper limit of the behaviour factors is equal for eccentrically braced frames and moment resisting frames of high ductility class (i.e., $q=5\alpha_u/\alpha_1$). However, the values resulting from this expression are generally different because of the global overstrength factors α_u/α_1 suggested for the two structural types. In fact, the reference value of the global overstrength factor varies from 1.1 to 1.3 for moment resisting frames while it is equal to 1.2 for eccentrically braced frames. According to some researchers (Mazzolani *et al*. 2009b) this choice of the global overstrength factor is questionable because the plastic redistribution capacity of the moment resisting frames is usually higher than that of the eccentrically braced frames. In this regard, the Authors note that the strain hardening of the links is, however, more significant than that of the dissipative zones of the moment resisting frames. Owing to this, the Authors observe that the value 1.2 of α_u/α_1 might be low (see Bosco *et* *al*. 2014) and that maybe an overestimation is made in the value 5 which defines the behaviour factor in the absence of any global overstrength factor.

In addition, the procedure described in Eurocode 8 for the explicit calculation of the global overstrength factor (by means of a nonlinear static analysis) is not easily applicable to eccentrically braced structures. In particular, the procedure for the calculation of the load factor α_u appears as to be conceived for moment resisting structures and then extended to eccentrically braced structures without specific checks. The value of α_u refers to the development of the overall structural instability and thus requires that the stiffness of the links tends to zero as the deformation tends to infinity. This hypothesis is not satisfied in common bilinear modelling of link beams because the overall hardening cannot be neglected. A suitable and simple model of the link should be multilinear, as proposed for example by Ramadan and Ghobarah (1995) but with a zero stiffness in the last branch. Still, to avoid an irregular use of the procedure intended for the calculation of the parameter α_u, a specific pattern of the lateral forces should be stipulated in the code (e.g., inverted triangular or proportional to the first mode of vibration of the structure).

6. Method of analysis

According to Eurocode 8, both the lateral force method of analysis and the modal response spectrum analysis may be applied for the design of eccentrically braced framed structures. The code restrictions to the use of the lateral force method of analysis do not depend on the structural type. As demonstrated in other papers (Rossi and Lombardo 2007, Bosco and Rossi 2009), the wide range of application of the lateral force method of analysis can strongly undermine the efficient performance of the eccentrically braced structures because the normalised link overstrength factors obtained by the use of this method of analysis, i.e., the ratios of the link overstrength factors to the minimum value of this parameter in the structure, may result in significant errors. In particular, at the upper storeys of medium and high-rise buildings the lateral force method of analysis is expected to provide overestimated values of the normalised overstrength factor of links. Under these circumstances, the design provisions intended to cause massive participation of links in the inelastic response of structures are likely to be unsuccessful and, therefore, the behaviour factors proposed for high ductility structures should not be allowed.

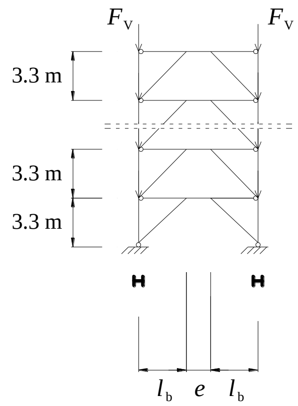


Fig. 2 Geometry and vertical loads of the exemplary eccentrically braced frame

Table 1 Exemplary case no. 1

Storey	Section			Ω/Ω_{min}			V_y/V_{Ed}		
	Links	Braces	Columns	LFMA	MRSA	Dyn	LFMA	MRSA	Dyn
8	HEA 160	HEM 100	HEA 180	1.23	1.00	1.00	1.28	0.92	0.75
7	HEB 180	HEM 100	HEA 180	1.06	1.06	1.08	1.14	1.01	0.81
6	HEB 220	HEM 140	HEB 240	1.02	1.18	1.29	1.10	1.13	0.97
5	HEB 260	HEM 140	HEB 240	1.04	1.29	1.53	1.12	1.23	1.15
4	HEB 280	HEM 160	HEB 300*	1.02	1.29	1.52	1.10	1.23	1.14
3	HEB 300	HEM 160	HEB 300*	1.04	1.29	1.48	1.11	1.23	1.11
2	HEB 300	HEM 160	HEB 500*	1.00	1.21	1.37	1.07	1.15	1.02
1	HEB 320	HEM 160	HEB 500*	1.04	1.24	1.38	1.11	1.18	1.02

steel grade S235; *steel grade S275

The importance of the method of analysis for the design of eccentrically braced structures is demonstrated here by means of an exemplary case consisting of an eight-storey braced frame founded on soft soil (type C according to Eurocode 8 2003). The geometric and mass properties of this frame are equal at all storeys and described in Fig. 2. The geometric length e of the links is equal to 0.1 times the length L of the braced span. The vertical load on the links and beam segments outside links is null because two beam members are assumed at each level of the eccentrically braced frame (EBF) (Perretti 1999, Rossi and Lombardo 2007) instead of the traditional single section: while the first section sustains the vertical loads transmitted by the deck, the second resists the horizontal actions and constitutes the link itself. To highlight the importance of the use of either the lateral force method of analysis or the modal response spectrum analysis the P-Δ effects are ignored. The frame is designed according to the procedure stipulated in Eurocode 8 and the internal forces on members are determined by the lateral force method of analysis (LFMA). The peak ground acceleration is equal to 0.35 g and a behaviour factor equal to 5 is considered. The cross-sections and steel grade of the members are reported in Table 1. As suggested in Eurocode 8, the plastic shear force and the plastic bending moment of the I-sections of the link beams are evaluated as

$$V_p = \frac{f_y}{\sqrt{3}} t_w (d - t_f) \tag{8}$$

$$M_p = f_y b \; t_f (d - t_f) \tag{9}$$

where f_y is the yield strength of steel, t_f and b are the thickness and width of the flange and t_w and d are the thickness and depth of the web.

The links are mechanically short at all storeys except for the top one where the links are intermediate length. In view of the internal forces resulting from the lateral force method of analysis (LFMA in the table), the link overstrength factors are everywhere lower than 1.25 times the minimum overstrength factor in the frame. This evaluation works out to be clearly erroneous if the internal forces are calculated by means of the modal response spectrum method of analysis (MRSA). These normalised overstrength factors are often remarkably different from those resulting from the lateral force method of analysis and characterised by a completely different distribution in elevation because of higher mode effects. In particular, the maximum value of the normalised overstrength factors Ω/Ω_{min} obtained by means of the modal response spectrum analysis is equal to 1.29 and thus higher than the limit value considered in Eurocode 8; also, this maximum normalised overstrength factor is obtained in the middle of the frame, on the contrary of what was anticipated by the use of the lateral force method of analysis. Still in Table 1 it is worth noting that the yield shear strength of the link adopted for the top storey is lower than the design shear force resulting from the modal response spectrum analysis. Finally, to compare the suitability of the two methods of analysis, the normalised overstrength factors are calculated based on the internal forces resulting from a linear time-history analysis of the frame (column "Dyn" in the table). The seismic input is represented by a set of ten accelerograms compatible with the elastic response spectrum proposed in Eurocode 8. The single accelerogram is defined by a stationary random process modulated by means of a trapezoidal intensity function. The total length of the accelerogram is equal to 30.5 s and that of the strong motion phase is 22.5 s. The Rayleigh formulation is used to introduce damping. Mass and stiffness coefficients are defined so that the first and third modes of vibration of the structures are characterised by an equivalent viscous damping factor equal to 0.05. The analyses are performed by the OpenSEES computer program. The comparison between the obtained results shows that the real overstrength factors are more scattered than those calculated according to both the design methods of analysis even if the MRSA provides results which are closer to the actual values.

7. *P*-Δ effects

In the past, researchers and seismic codes indicated the strategy based on the increase in the structural strength as the most suitable to counterbalance the effects of gravity loads on the displacement demand of the system. The required amplification of the structural strength is generally quantified by means of the interstorey drift sensitivity coefficient θ. The coded values of this amplification are expected to ensure that the first order displacement demand of the system designed to resist the design base shear and the second order displacement demand of the system designed to resist the amplified design base shear are equal. Although this goal is subscribed by codes, the mathematical expression of the coefficient θ reported in Eurocode 8 (Elghazouli 2007, 2010, Peres and Castro 2010, Peres 2010) leads to values which are much higher than those derived from recent American codes (FEMA 2003, ASCE 2010) and the design provisions proposed in Eurocode 8 in regard to *P*-Δ effects are generally more exacting than those considered in other codes (Amara *et al*. 2014).

The mathematical expression of the interstorey drift sensitivity coefficient (Eq. (7)) is formally the same in Eurocode 8 as in the American codes. However, in the abovementioned codes the interstorey displacement considered for the evaluation of the interstorey drift sensitivity coefficient is given a different meaning. In the American codes the interstorey displacement is intended as the elastic interstorey displacement experienced by the structure subjected to the design seismic actions; in Eurocode 8, instead, the interstorey displacement is an estimate of the inelastic displacement (see Section 3, Eq. (7)).

To comprehend more in depth the conservatism of the design provisions of Eurocode 8,

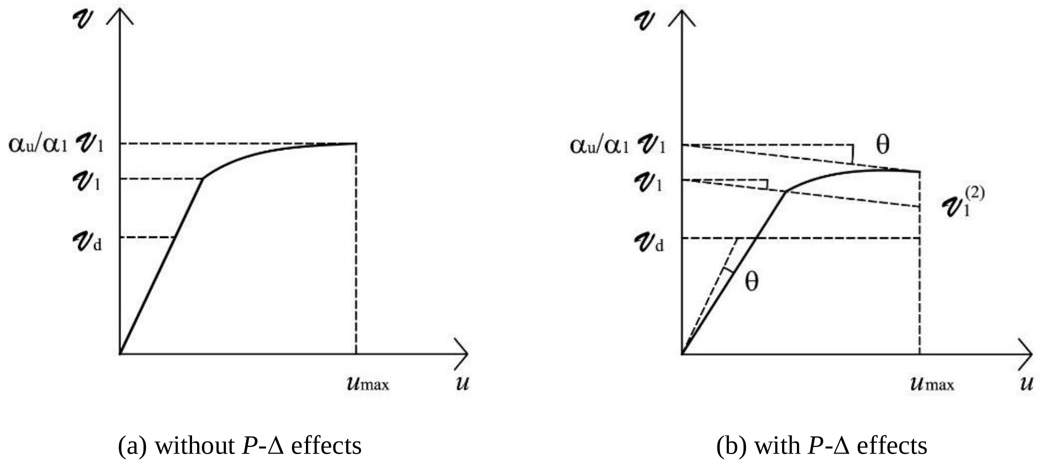


Fig. 3 Capacity curve

consider a system designed to resist a design base shear force $\mathcal{V}_d$. The system is characterised by a yield storey shear strength equal to $\mathcal{V}_1$ in the absence of *P*-Δ effects and is endowed with an ultimate storey strength equal to $\alpha_u/\alpha_1 \mathcal{V}_1$. The capacity curve of this system, i.e., the relationship between the base shear force and the top horizontal displacement, is plotted schematically in Fig. 3(a).

To counterbalance the effects of gravity loads on the displacement demand, the yield storey shear strength $\mathcal{V}_1$ must be higher than an amplified design shear force $\mathcal{V}_d^{(+)}$ which is given by the equation

$$\mathcal{V}_d^{(+)} = \frac{1}{1-\theta_d}\mathcal{V}_d \tag{10}$$

where θ_d is named here *design interstorey drift sensitivity coefficient*. This parameter is different from the interstorey drift sensitivity coefficient commonly considered in research studies on SDOF systems (Bernal 1987, Panchia 1989, Fenwick *et al*. 1992, MacRae 1994, Tremblay *et al*. 1998, Humar *et al*. 2006) because it does not identify the amplification of the ultimate shear strength of the system. Rather, the design interstorey drift sensitivity coefficient θ_d quantifies the amplification of the design shear force and applies to the design seismic internal forces. In the evaluation of the design interstorey drift sensitivity coefficient θ_d reported in Eurocode 8 (later indicated as θ_{EC8}), the horizontal displacement is set equal to the inelastic design displacement u_{max} while the corresponding shear force is assumed equal to the design value $\mathcal{V}_d$, i.e., Eurocode 8 assumes that the structure has an ideal elastic-perfectly plastic behaviour characterized by a yielding force equal to $\mathcal{V}_d$. Owing to this, any structural overstrength introduced in the process of design is neglected.

Note that if the system is provided with a yield storey shear strength $\mathcal{V}_1^{(+)} \geq \mathcal{V}_d^{(+)}$, the shear force $\mathcal{V}_1^{(2)}$ sustained by the system at the maximum inelastic displacement taking into account *P*-Δ effects is not lower than the design shear force $\mathcal{V}_d$ (Fig. 3(b)). Therefore, the following inequality must be satisfied

$$\mathcal{V}_1^{(2)} \geq \mathcal{V}_\mathrm{d} \tag{11}$$

Neglecting the structural overstrength makes the approach of Eurocode 8 for the evaluation of θ_d conservative. It is opinion of the Authors that the overstrength could be taken into account and thus that the parameter θ_d could be calculated by substituting the design shear force $\mathcal{V}_\mathrm{d}$ for the ultimate strength $\alpha_u/\alpha_1 \mathcal{V}_1$. The proposed expressions of the design interstorey drift sensitivity coefficient are derived below. Two cases are separately considered because in Eurocode 8 the inelastic displacement d_r is calculated as the elastic design interstorey displacement times the displacement behaviour factor q_d and, in turn, the displacement behaviour factor is calculated according to two expressions as a function of the fundamental period of vibration T of the system. Specifically, if the fundamental period of vibration T of the system is higher than the upper limit T_C of the period of the constant spectral acceleration branch the displacement behaviour factor q_d is assumed equal to the behaviour factor q; if the period of vibration T is lower than T_C, instead, the ductility demand is expected to be higher than the behaviour factor and the displacement behaviour factor is calculated as $q_\mathrm{d}=1+(q-1)T_C/T$.

7.1 Long period systems

The relationship between $\mathcal{V}_1^{(+)}$ and $\mathcal{V}_1^{(2)}$ (likewise that between $\alpha_\mathrm{u}/\alpha_1 \mathcal{V}_1^{(+)}$ and $\alpha_\mathrm{u}/\alpha_1 \mathcal{V}_1^{(2)}$) should be calculated by considering that, in the absence of P-Δ effects, the shear strength corresponding to the inelastic design displacement is $\alpha_u/\alpha_1 \mathcal{V}_1$. Thus, if the period of vibration T is higher than T_C, the commonly adopted relationship between $\mathcal{V}_1^{(+)}$ and $\mathcal{V}_1^{(2)}$ is given by the equation

$$\frac{\mathcal{V}_1^{(2)}}{1-\theta_\mathrm{EC8}/\left(\bar{\alpha}\bar{\mathcal{V}}_1\right)} = \mathcal{V}_1^{(+)} \tag{12}$$

where $\bar{\alpha} = \alpha_\mathrm{u}/\alpha_1$, $\bar{\mathcal{V}}_1 = \mathcal{V}_1/\mathcal{V}_\mathrm{d}$, $\bar{\mathcal{V}}_1^{(2)} = \mathcal{V}_1^{(2)}/\mathcal{V}_\mathrm{d}$ and θ_EC8 is the interstorey drift sensitivity coefficient evaluated according to Eurocode 8.

Equating Eq. (10) and Eq. (12) provides the following expression of the design interstorey drift sensitivity coefficient

$$\theta_\mathrm{d} = 1 - \frac{1}{\bar{\mathcal{V}}_1^{(2)}}\left(1 - \frac{\theta_\mathrm{EC8}}{\bar{\alpha}\bar{\mathcal{V}}_1}\right) \tag{13}$$

The value of the design interstorey drift sensitivity coefficient leading to the lowest amplification of the design shear force is that corresponding to $\bar{\mathcal{V}}_1^{(2)}=1$. This value, suggested here for design instead of that proposed in Eurocode 8, is

$$\theta_\mathrm{d} = \frac{\theta_\mathrm{EC8}}{\bar{\alpha}\bar{\mathcal{V}}_1} \tag{14}$$

and the corresponding amplification of the design shear force is

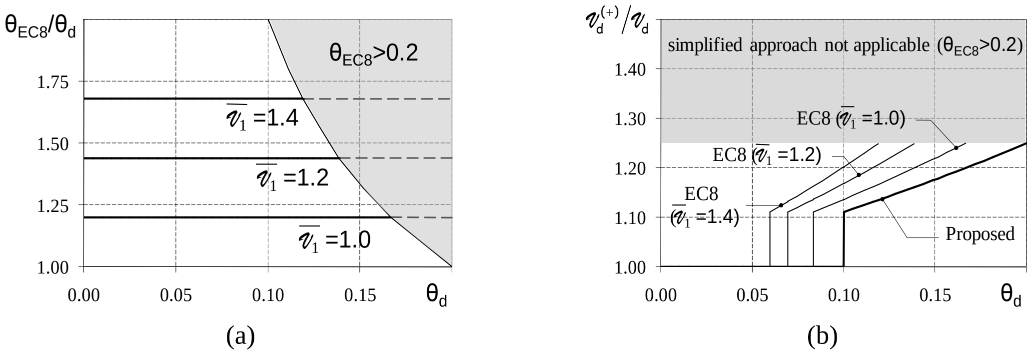


Fig. 4 Long period systems ($T \geq T_C$): (a) ratio of the interstorey drift sensitivity coefficients, (b) amplification of the design shear force

$$\bar{\vartheta}_d^{(+)} = \frac{1}{1 - \frac{\theta_{EC8}}{\bar{\alpha}\bar{\vartheta}_1}} \tag{15}$$

This value of the interstorey drift sensitivity coefficient is lower than θ_{EC8} if the ratio α_u/α_1 is higher than unity. In particular, if the ratio α_u/α_1 is equal to 1.2 (as generally assumed for eccentrically braced structures in Eurocode 8) and $\bar{\vartheta}_1$ =1, the interstorey drift sensitivity coefficient θ_d is equal to about 0.83 θ_{EC8}.

The simplified approach for the evaluation of the *P*-Δ effects is suitable for counteracting the abovementioned *P*-Δ effects provided that the overturning moment due to *P*-Δ effects corresponding to the inelastic displacement is not higher than 20% of the bending moment produced by the seismic shear forces in a first order structural analysis. This means that θ_d must be not higher than 0.20.

Assuming θ_d=$\theta_{d,max}$=0.20 in Eq. (14), the value of the interstorey drift sensitivity coefficient θ_{EC8} corresponding to the abovementioned limit is obtained

$$\theta_{EC8,\,max} = 0.20\,\bar{\alpha}\bar{\vartheta}_1 \tag{16}$$

The relationship between the ratio θ_{EC8}/θ_d and the design interstorey drift sensitivity coefficient θ_d is illustrated in Fig. 4(a). The values of the ratio θ_{EC8}/θ_d are derived from Eq. (14) and plotted for different values of the normalised yield shear strength $\bar{\vartheta}_1$. The lines in the figure are solid up to a value of θ_{EC8} equal to 0.20 while they are dashed for values of θ_{EC8} which are higher than 0.20 but not greater than $\theta_{EC8,max}$ (Eq. (16)). Note that the parameter $\theta_{EC8,max}$ depends on the normalised yield shear strength $\bar{\vartheta}_1$ and ratio α_u/α_1. The maximum values of the parameter $\theta_{EC8,max}$ are equal to 0.240, 0.288 and 0.336 for values of $\bar{\vartheta}_1$ equal to 1.0, 1.2 and 1.4, respectively. In particular, values of θ_{EC8} within the dashed segments correspond to structural systems in which *P*-Δ effects can be considered by means of the simplified approach only if the proposal of the authors is accepted. This may be of help in medium and high-rise eccentrically braced structures where the lateral stiffness (and generally the lateral strength) is often increased, with respect to the value deriving from the no-collapse requirement, to satisfy the damage limitation requirement.

The normalised shear strength $\bar{\mathcal{V}}_{\mathrm{d}}^{(+)}$ is reported in Fig. 4(b). The thick line identifies the normalised shear strengths proposed by the authors, i.e., values which are equal to unity for $\theta_d<0.10$ and calculated by Eq. (15) for $0.10\leq\theta_d\leq0.20$. The other lines identify normalised shear strengths calculated by Eurocode 8 for different values of the normalised yield shear strength $\bar{\mathcal{V}}_1$. In these cases, $\bar{\mathcal{V}}_{\mathrm{d}}^{(+)}$ is equal to unity for $\theta_{EC8}<0.10$ and equal to $\bar{\mathcal{V}}_{\mathrm{d}}^{(+)}=1/(1-\theta_{EC8})$ for $0.10\leq\theta_{EC8}\leq0.20$.

7.2 Short period systems

The equations reported in the previous section are derived here with reference to the case in which the fundamental period of vibration of the system is lower than T_C. As stated before, in this case the displacement behaviour factor is calculated as $q_d=1+(q-1)T_C/T$. If this expression of the displacement behaviour factor is used, the relationship between $\mathcal{V}_1^{(+)}$ and $\mathcal{V}_1^{(2)}$ (likewise that between $\alpha_u/\alpha_1\mathcal{V}_1^{(+)}$ and $\alpha_u/\alpha_1\mathcal{V}_1^{(2)}$) becomes

$$\frac{\mathcal{V}_1^{(2)}}{1-\theta_e\left[1+\left(q\dfrac{\mathcal{V}_d}{\alpha_u/\alpha_1\mathcal{V}_1}-1\right)\dfrac{T_C}{T}\right]}=\mathcal{V}_1^{(+)} \tag{17}$$

where $\theta_e=\theta_{EC8}/q_d$ is the interstorey drift sensitivity coefficient corresponding to the elastic design interstorey displacement and is later named *elastic interstorey drift sensitivity coefficient*.

Equating Eq. (10) and Eq. (17) provides the following expression of the design interstorey drift sensitivity coefficient

$$\theta_d=1-\frac{1}{\bar{\mathcal{V}}_1^{(2)}}\left\{1-\theta_e\left[1+\left(\frac{q}{\bar{\alpha}\bar{\mathcal{V}}_1}-1\right)\bar{T}\right]\right\} \tag{18}$$

where $\bar{T}=T_C/T$. The value of θ_d leading to the lowest amplification of the design shear force is

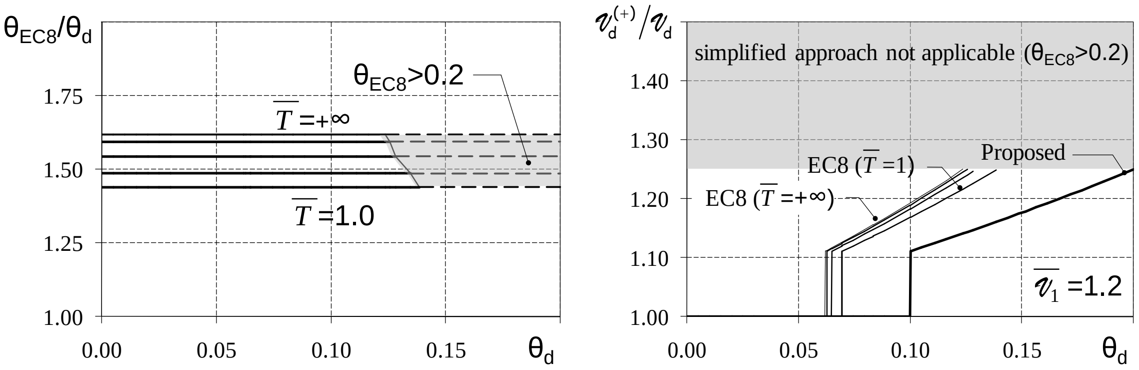
$$\theta_d=\frac{\theta_{EC8}}{q_d}\left[1+\left(\frac{q}{\bar{\alpha}\bar{\mathcal{V}}_1}-1\right)\bar{T}\right] \tag{19}$$

The normalised shear strength $\bar{\mathcal{V}}_{\mathrm{d}}^{(+)}$ corresponding to this last value of θ_d is

$$\bar{\mathcal{V}}_d^{(+)}=\frac{1}{1-\dfrac{\theta_{EC8}}{q_d}\left[1+\left(\dfrac{q}{\bar{\alpha}\bar{\mathcal{V}}_1}-1\right)\bar{T}\right]} \tag{20}$$

Also in this case the simplified approach may be applied if θ_d is not greater than 0.20, i.e., if the interstorey drift sensitivity coefficient θ_{EC8} is not greater than

$$\theta_{EC8,\,max}=0.20\,\frac{q_d}{1+\left(\dfrac{q}{\bar{\alpha}\bar{\mathcal{V}}_1}-1\right)\bar{T}} \tag{21}$$



(a) (b)

Fig. 5 Short period systems ($T<T_C$): (a) ratio of the interstorey drift sensitivity coefficients, (b) amplification of the design shear force

The relationship between the ratio θ_{EC8}/θ_d and the interstorey drift sensitivity coefficient θ_d is illustrated in Fig. 5(a) assuming q=5, $\bar{\alpha}=1.2$, $\bar{\mathcal{V}}_1=1.2$ and for values of the parameter $\bar{T}$ equal to 1, 1.5, 3, 10 and +∞; in particular, the case $\bar{T}$ =1 is reported in the figure to allow an immediate comparison with the values of θ_{EC8}/θ_d corresponding to long period systems. As is evident, the ratio θ_{EC8}/θ_d slightly increases with $\bar{T}$. The relationship between the design interstorey drift sensitivity coefficient θ_d and the amplified design shear force $\bar{\mathcal{V}}_d^{(+)}$ is illustrated in Fig. 5(b). In the same figure is also plotted, for comparison, the amplified design shear force required by Eurocode 8.

8. Links

In the context of the design procedure proposed in Eurocode 8 for eccentrically braced structures, the limitation of the normalised link overstrength factor seems to reveal the intention of the code to aim at a homogeneous dissipative behaviour of the links by means of the design provisions proposed by Popov *et al*. (1992) in the early 1990s (Popov *et al*. 1992, Lu *et al*. 1997, Kasai and Han 1997). The American researchers proposed to favour a simultaneous yielding of links of all floors in the belief that this could ensure uniform damage of the links along the height of the building. To this end, Popov *et al*. (1992) defined the link overstrength factor as the ratio of the link yield shear strength to the link design shear force and recommended that these factors were kept nearly uniform in elevation. Despite the apparent intention of the European seismic code, the design provisions reported in this code do not comply entirely with those proposed by Popov *et al*. (1992) In Eurocode 8, in fact, the link overstrength factor is defined with reference to the ultimate internal forces of the links, i.e., this factor calculates how much the ultimate link strength exceeds the design internal force. This is evident in Eqs. (3)-(4) where the products $1.5V_p$ and $1.5M_p$ are approximate evaluations of the ultimate strengths of the links (Malley and Popov 1983, 1984, Hjelmstad and Popov 1984, Kasai and Popov 1986a, b, Engelhardt and Popov 1989a, Della Corte *et al.* 2013). But the ratio of the ultimate strengths of two generic links does not

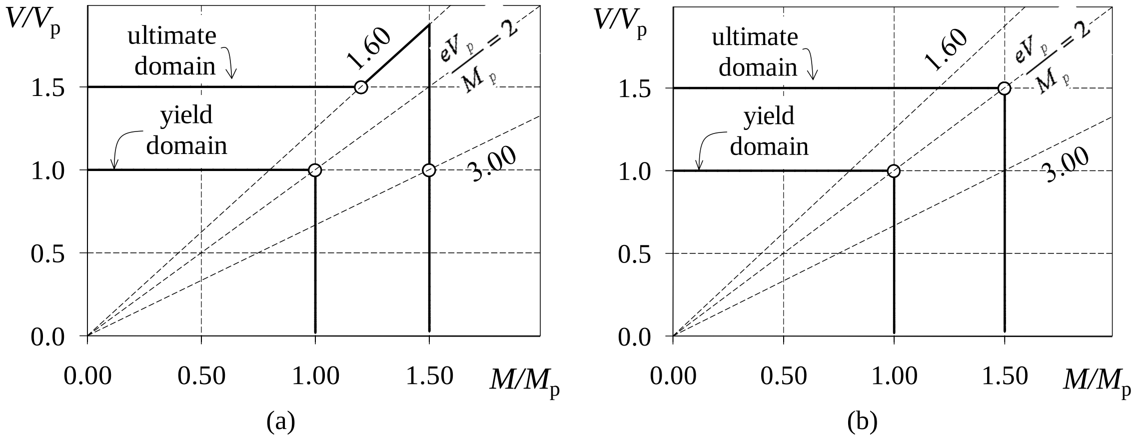


Fig. 6 Yield and ultimate domain of links according to (a) Eurocode 8; (b) null interaction between bending moment and shear force

always equal the ratio of their yield strengths or, in other words, the yield and ultimate strength domains of links are not always proportional. As can be easily deduced by the design provisions reported in previous sections, Eurocode 8 considers that the yield internal forces of links of eccentrically braced frames in the *K*-split geometric configuration can be calculated as (see Fig. 6)

$$V_{y,i} = V_{p,i} \qquad M_{y,i} = eV_{p,i}/2 \qquad \text{if } eV_{p,i}/M_{p,i} \le 2.0 \tag{22}$$

$$V_{y,i} = 2M_{p,i}/e \qquad M_{y,i} = M_{p,i} \qquad \text{if } eV_{p,i}/M_{p,i} > 2.0 \tag{23}$$

and that the ultimate internal forces may be assumed as

$$V_{u,i} = 1.5V_{p,i} \qquad M_{u,i} = eV_{u,i}/2 \qquad \text{if } eV_{p,i}/M_{p,i} \le 1.6 \tag{24}$$

$$V_{u,i} = 2M_{u,i}/e \qquad M_{u,i} = 1.5M_{p,i} \qquad \text{if } eV_{p,i}/M_{p,i} > 1.6 \tag{25}$$

As is apparent, the mathematical expressions of the ultimate internal forces are proportional to those of the yield internal forces but the range of application of Eqs. (22)-(23) is different from that of Eqs. (24)-(25). Owing to this, the ratio of the overstrength factors calculated by Eqs. (3)-(4) does not always equal the ratio of the overstrength factors considered in (Popov *et al*. 1992, Lu *et al*. 1997, Kasai and Han 1997). In particular, ultimate and yield domains are not proportional in the range of the mechanical lengths from 1.6 to 2.0.

Moreover, as also reported by other researchers (Mazzolani *et al*. 2009b), the link overstrength factor defined in Eurocode 8 is discontinuous at a specific value of the mechanical link length eV_p/M_p because of a discontinuity in the ultimate shear force considered in Eqs. (24)-(25). Such an anomaly is unjustified and causes abrupt variations in the link resistance when the mechanical length of this member is moved up and down the point of discontinuity. Specifically, in eccentrically braced frames characterised by the *K*-split geometric configuration the ultimate shear force of the links is discontinuous at a value of eV_p/M_p equal to 1.6 (see Fig. 6).

The effect of the non-proportionality of yield and ultimate domains of links is shown here with reference to a frame characterised by twelve storeys and normalised link length e/L equal to 0.15.

Table 2 Exemplary case no. 2

Storey	Section			$\frac{eV_p}{M_p}$	Ω/Ω_{min}	Ω/Ω_{min}
	Links	Braces	Columns		EC8	expected
12	HEB 180	HEM 120	HEA 180	2.337	1.20	1.29
11	HEB 200	HEM 120	HEA 180	2.078	1.04	1.12
10	HEB 220	HEM 120	HEB 240	1.870	1.11	1.13
9	HEB 220	HEM 120	HEB 240	1.870	1.05	1.08
8	HEB 240	HEM 140	HEM 240*	1.698	1.21	1.12
7	HEB 240	HEM 140	HEM 240*	1.698	1.15	1.06
6	HEB 240	HEM 140	HEM 260*	1.698	1.08	1.00
5	HEB 280	HEM 140	HEM 260*	1.443	1.02	1.09
4	HEB 300	HEM 160	HEM 280**	1.337	1.04	1.11
3	HEB 320	HEM 160	HEM 280**	1.296	1.06	1.14
2	HEB 320	HEM 180	HEM 300**	1.296	1.00	1.08
1	HEB 340	HEM 180	HEM 300**	1.289	1.03	1.11

steel grade S235; * steel grade S275; ** steel grade 355

As for the frame depicted in Fig. 2, the geometric and mass properties of this frame are equal at all storeys. Again, the *P*-Δ effects are not taken into account and the vertical load on the links and beam segments outside links is assumed to be null because of the presence of two beam members at each level of the frame. The system is founded on soft soil (type C) and designed by modal response spectrum analysis. The cross-sections and steel grade of the members are reported in Table 2 along with the mechanical length of the links. The links are mechanically short at the first five storeys and intermediate length at the remaining storeys. The normalised overstrength factors calculated according to Eurocode 8 (Ω/Ω_{min}-EC8 in the table) are always lower than 1.25. To show the influence of the aforementioned discontinuity on the design of eccentrically braced frames, the overstrength factors are also calculated in the manner of Popov *et al*. (1992) (i.e., as V_y/V_{Ed}) and the ratios of these overstrength factors to their minimum value in the frame are shown in the table (*expected* Ω/Ω_{min}). As is evident from the comparison of the two different evaluations of the normalised overstrength factors, the definition proposed in Eurocode 8 may bring about some errors in the design of eccentrically braced frames. However, it is right to say that the importance of the aforementioned discontinuity in the design of eccentrically braced structures is generally not remarkable and that the identification of a case in which the effect of this error is evident is neither simple nor immediate. Further, the difference between the values resulting from the two definitions of the normalised overstrength factor is never striking. Despite this consideration, it is worth noting that in the literature some simple relationships of the ultimate shear force of the links are present which do not cause discontinuity in the link overstrength factor, e.g., (Richards and Uang 2002, Bosco and Rossi 2009). As an example, Fig. 6 shows the ultimate plastic domain corresponding to the equations reported in Eurocode 8 and that corresponding to null interaction between bending moment and shear force. As demonstrated by numerous laboratory tests (Kasai and Popov 1986c), this latter domain is slightly conservative. Nonetheless, it is particularly favourable in design because of the simple mathematical expressions that define the ultimate shear force and bending moment. These ultimate internal forces are calculated by means of the following expressions

$$V_{\text{u,i}} = 1.5V_{\text{p,i}} \qquad M_{\text{u,i}} = eV_{\text{u,i}}/2 \qquad \text{if} \;\; eV_{\text{p,i}}/M_{\text{p,i}} \le 2.0 \tag{26}$$

$$V_{\text{u,i}} = 2M_{\text{u,i}}/e \quad M_{\text{u,i}} = 1.5M_{\text{p,i}} \qquad \text{if} \;\; eV_{\text{p,i}}/M_{\text{p,i}} > 2.0 \tag{27}$$

In Eurocode 8 the internal forces considered for the evaluation of the link overstrength factor comprise the contribution specified to counterbalance *P*-Δ effects. As reported in Eq. (7), these latter forces are evaluated with reference to the maximum inelastic displacements of the structure and are, therefore, not entirely required at the first yielding of links. As not proportional to the seismic internal forces obtained by the design structural analysis, these internal forces may lead to rough evaluations of the overstrength factors at the first yielding of links. To eliminate this error, some modification of the overstrength factor should also be devised which allows the evaluation of the internal forces effectively required at first yielding of links because of *P*-Δ effects.

As demonstrated by Elghazouli (2010) with regard to moment resisting steel structures, the mathematical formulation of the overstrength factor reported in Eurocode 8 ignores the internal forces caused by gravity loads. It should be noted that in the case of the eccentrically braced structures this error can be null if two beams are considered instead of the single beam at each storey of the eccentrically braced frames (Rossi 2007) because in this case gravity loads are not applied to links. Aside from this particular case, a moderate influence of the gravity loads on the link overstrength factor cannot be denied in general.

To appraise the influence of the vertical loads on the overstrength factor of links, a parametric analysis is carried out on one-storey systems characterised by link length from 0.01*L* to 0.50*L*. The gravity loads G are uniformly applied on the link as well as on the adjacent beam segments: the magnitude of these loads ranges from 0 to 40 kN/m in the seismic design situation and from 0 to 73.2 kN/m in the non-seismic design situation.

Seismic loads are represented by a horizontal force applied to the deck of the one-storey system. The magnitude of this force is assumed as the storey mass (146.8*t*) times a pseudo-acceleration ranging from 0.05 g to 0.35 g. This should allow the analysis of the behaviour of links belonging to either upper or lower storeys of multi-storey systems and undergoing moderate to high seismic forces. Specifically, the lower values of the pseudo-accelerations are used to reproduce the seismic effects of upper storeys of structures, while the higher values are used to simulate the response of lower storeys. The link shear force $V_{\text{Ed,E}}$ and the bending moment $M_{\text{Ed,E}}$ due to seismic actions are constant and proportional to the link length, respectively. The shear force $V_{\text{Ed,G}}$ and the bending moment $M_{\text{Ed,G}}$ caused by gravity loads at the ends of the links can be evaluated by the following simple equations assuming that the link and the adjacent beam segments can be schematised as a continuous beam resisting on three spans

$$V_{\text{Ed,G}} = G\frac{e}{2} \tag{28}$$

$$M_{\text{Ed,G}} = \frac{G\left(l_b^3 + e^3\right)}{8\cdot\left(l_b + 3/2\;\; e\right)} = \frac{GL^2}{4\cdot(1+2e/L)}\cdot\left[\frac{7}{8}\left(\frac{e}{L}\right)^3 + \frac{3}{8}\left(\frac{e}{L}\right)^2 - \frac{3}{8}\frac{e}{L} + \frac{1}{8}\right] \tag{29}$$

where l_b is the length of the beam element outside the link.

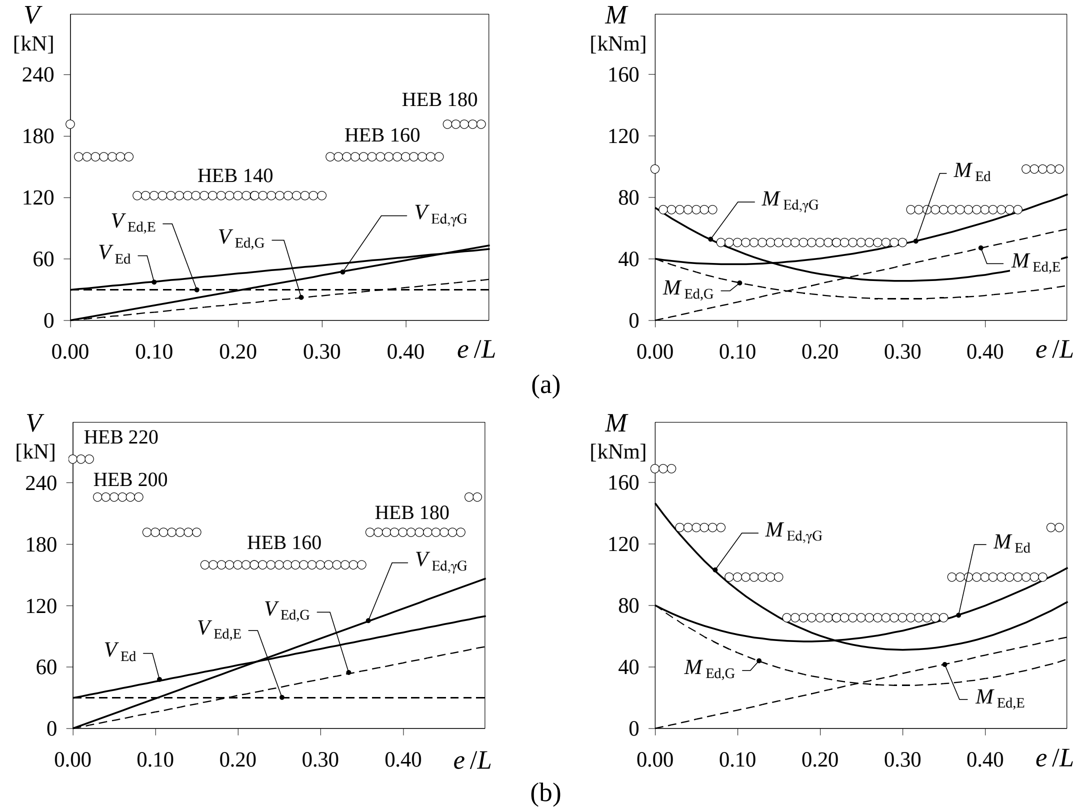


Fig. 7 Design shear forces and bending moments for moderate seismic loads: (a) a_g=0.05 g; G=20 kN/m; (b) a_g=0.05 g; G=40 kN/m

These equations show that the shear force produced by gravity loads is proportional to the link length and generally moderate in value. The bending moment, instead, is nonlinear with the length of the link and adjacent beam segments; it is higher in short links than in long links and gains its minimum value for a normalised link length e/L approximately equal to 0.30.

The shear force and the bending moment caused by gravity and seismic forces on the link and beam segments of the abovementioned single-storey system are reported in Figs. 7 and 8. In these figures the solid lines refer to the internal forces caused by the seismic forces and gravity loads of the seismic design situation (V_{Ed}, M_{Ed}) and to the internal forces due to the gravity loads of the non-seismic design situation ($V_{Ed,\gamma G}$, $M_{Ed,\gamma G}$). The dashed lines indicate the contributions of the seismic ($V_{Ed,E}$, $M_{Ed,E}$) and gravity loads ($V_{Ed,G}$, $M_{Ed,G}$) to the internal forces in the seismic design situation. Also reported in the figures (circles) are the resisting internal forces of the adopted link sections.

In the case of moderate seismic loads (Fig. 7), the design of geometrically short links is governed by the bending moment $M_{Ed,\gamma G}$ caused by the loads of the non-seismic design situation. The maximum link length for which this occurs depends on the magnitude of the gravity loads. In the case of moderate gravity loads (see Fig. 7(a)) the design is governed by the non-seismic design situation if the link length is lower than $0.15L$; in the case of high gravity loads (see Fig. 7(b)) this limit length moves to about $0.22L$.

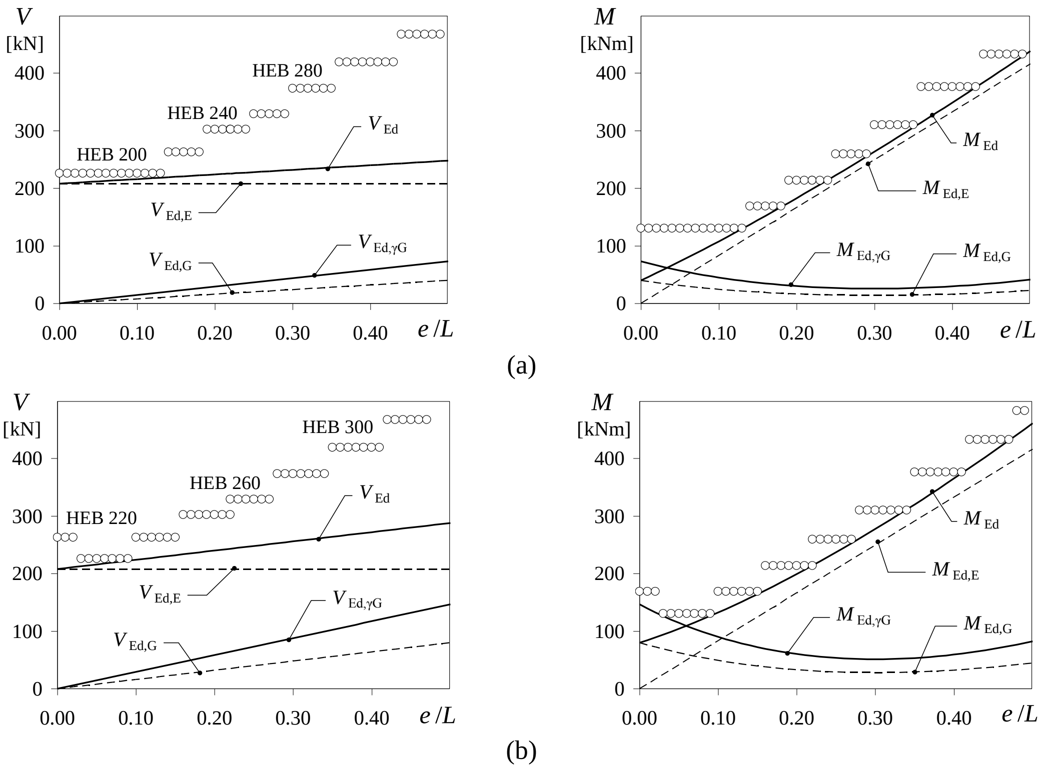


Fig. 8 Design shear forces and bending moments for important seismic loads: (a) a_g=0.35 g; G=20 kN/m; (b) a_g=0.35 g; G=40 kN/m

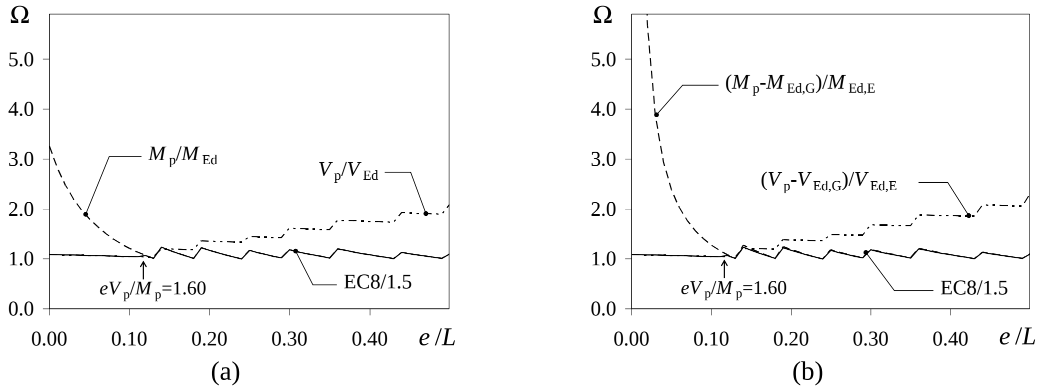


Fig. 9 Overstrength factors calculated by means of the (a) total internal forces in the seismic design situation (b) seismic internal forces (a_g=0.35 g; G=20 kN/m)

In the event of important seismic actions (Fig. 8), with the only exception of very short links subjected to high gravity loads (see Fig. 8(b)), the design of links is governed by the seismic design situation. Because of gravity loads, also in links with mechanical link length lower than 2 the verification for flexural strength may be more stringent than that for shear strength. The mechanical length corresponding to simultaneous yielding for flexure and shear may be calculated as

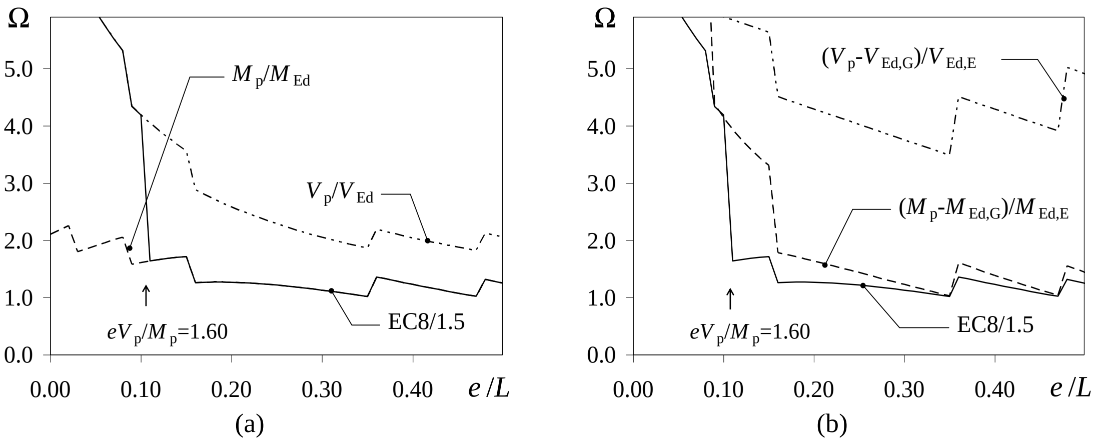


Fig. 10 Overstrength factors calculated by means of the (a) total internal forces in the seismic design situation (b) seismic internal forces. (a_g=0.05 g; G=40 kN/m)

$$\frac{eV_p}{M_p} = 2 \cdot \left(1 + \frac{eV_{Ed,G} - 2M_{Ed,G}}{2M_p} \right) \tag{30}$$

As is evident in Figs. 7 and 8, the design of long links is always governed by the bending moment in the seismic design situation.

Based on these considerations, the link overstrength factors are not correctly evaluated if gravity loads are ignored. The errors are evident in Figs. 9 and 10 where (i) the flexural overstrength factor of the link (dashed line), (ii) the shear overstrength factor of the link (dash-dotted line) and (iii) the link overstrength factor calculated according to Eurocode 8 by means of Eqs. (3)-(4) (solid line) are compared. Note that the real link overstrength factor is equal to the minimum between the flexural and shear overstrength factors of the link and that, to make the overstrength factors comparable, the link overstrength factor calculated according to Eurocode 8 is divided by 1.5. To highlight the importance of the gravity loads, the flexural and shear overstrength factors are first evaluated in Figs. 9(a) and 10(a) as M_p/M_{Ed} and V_p/V_{Ed} and thus as a function of the internal forces due to both gravity and seismic forces. Then, in Figs. 9(b) and 10(b), the same overstrength factors are evaluated correctly by means of the ratios $(M_p\text{-}M_{Ed,G})/M_{Ed,E}$ and $(V_p\text{-}V_{Ed,G})/V_{Ed,E}$.

In the case of moderate gravity loads (Fig. 9(a) and 9(b)), the use of the total internal forces in the seismic design situation leads to errors only in the flexural overstrength factor of short links. In fact, the curves corresponding to M_p/M_{Ed} and $(M_p\text{-}M_{Ed,G})/M_{Ed,E}$ significantly differ only in the left side of the plot. The curves referring to V_p/V_{Ed} and $(V_p\text{-}V_{Ed,G})/V_{Ed,E}$, instead, are almost coincident. In the case of the high gravity loads (Fig. 10(a) and 10(b)), instead, the errors are great for both the flexural and shear overstrength factors of short, intermediate and long links.

The overstrength factor of Eurocode 8 is based on the shear overstrength factor if $eV_p/M_p \leq 1.6$ and on the flexural overstrength factor if $eV_p/M_p > 1.6$. Further, these factors are calculated on the basis of the total internal forces in the seismic design situation. Owing to this, the evaluations of Eurocode 8 may be considered to be correct if the magnitude of the gravity loads is moderate but erroneous in the case of high gravity loads.

Further, it is worth noting that the correct evaluation of the overstrength factor is also hindered by the use of the design spectrum proposed in Eurocode 8. This spectrum is, in fact, not strictly proportional to the elastic response spectrum in the entire range of interest of the periods of vibration of the structures and alters the correct evaluation of the internal forces corresponding to the first yielding of links. This consideration applies to structures for which the pseudo-acceleration corresponding to the fundamental period of vibration equals the threshold minimum value considered in Eurocode 8, i.e., 0.2 a_g. This aspect is discussed in depth in reference by Bosco and Rossi (2009).

9. Braces, columns and beam segments outside links

As is evident in the Eq. (6), the seismic contributions to the design internal forces of the members intended for the elastic behaviour are assumed to be proportional to the internal forces deriving from the structural analysis of the system undergoing seismic actions. The factor which amplifies these latter forces is equal to 1.1 γ_{ov} times the minimum value of the link overstrength factor in the building. Setting aside the term 1.1 γ_{ov}, the seismic contributions to the design internal forces of the non-dissipative members are obtained supposing that (i) the ultimate internal forces are reached in the link with the minimum overstrength factor and that (ii) the design values of the seismic internal forces are in the same proportion as the seismic internal forces corresponding to the elastic behaviour of members. Again, in the Eq. (6) the minimum overstrength factor of the links is calculated as in Eqs. (3)-(4) while it should be calculated to represent the minimum ratio between the yield shear strength of the link and the design shear force.

The approach to the evaluation of the internal forces of the non-dissipative members is the same for all the steel structural types considered in Eurocode 8 and is strikingly different from that suggested with regard to reinforced concrete structures. In the latter case, the internal forces of the elements intended for the elastic behaviour are basically derived from the plastic strength of the dissipative members supposing that the ultimate deformation capacity is reached in every dissipative member. In the case of steel structures, instead, the application of the capacity design rules is more relaxed and the non-dissipative members are designed supposing that only some links reach the ultimate deformation capacity during the design ground motion. The method is easy to apply and favours the reduction of the structural costs. Despite these advantages, the reasonableness of the approach has been disputed by many researchers (e.g., Rossi and Lombardo 2007, Elghazouli 2007, 2010, Mazzolani *et al*. 2009b) because the real inelastic response of the eccentrically braced systems does not necessarily proceed in the same manner as the elastic one and thus the heightwise distribution of the inelastic internal forces of the links does not necessarily resemble the distribution of the elastic internal forces. As demonstrated by Bosco and Rossi (2009), this objection is fairly well-grounded with reference to eccentrically braced structures because the inelastic seismic response of these structures depends on both the overstrength factor of the links and the plastic redistribution capacity of the system.

As is apparent in Eq. (6) the internal forces corresponding to the first achievement of the ultimate plastic rotation in the links are increased by means of the two factors 1.1 and γ_{ov}. The meaning of the amplifying coefficient 1.1 is not clearly stated in Eurocode 8. According to some researchers (Elghazouli 2007) it is introduced to consider the strain hardening of steel and strain rate effects. The Authors note that, if this were true, the design provisions reported in Eurocode 8 would be at least partially inconsistent. In fact, the effect of the strain hardening of links is included in the factor 1.5 presently considered in the parameter **Ω**. Because of this, the amplifying factor 1.1 considered in the design of the eccentrically braced structures would be partly unjustified.

The parameter γ_{ov} is equal to the ratio of the expected (average) yield stress of steel to the nominal yield value and is introduced in Eq. (6) to reduce to acceptable values the probability that yielding or instability of non-dissipative members occurs prior to link failure. The presence of this parameter is essential because both the plastic (or buckling) strength of the non-dissipative members and the maximum seismic internal forces nominally transmitted to these elements by links are calculated on the basis of an equal fractile of the steel strength (Rossi and Lombardo 2007). The Eurocode 8 suggests using a value of the material overstrength factor γ_{ov} equal to 1.25 independently of the yield strength of steel but states that National Authorities have the freedom to select more appropriate values. In this regard, some researchers note that this value should be better differentiated depending on the yield strength of the steel adopted for the members, as also reported in some codes (Italian building code 2008) and justified in the literature (Rossi and Lombardo 2007).

A modification to the approach proposed in Eurocode 8 for the evaluation of the internal forces of the non-dissipative members has recently been suggested by Elghazouli (2007) with reference to moment resisting structures. The adjustment aims at more prudent estimates of the design internal forces of these members and consists in multiplying the minimum value of the overstrength factor $\mathbf{\Omega}_{min}$ by the global overstrength factor α_u/α_1. According to this proposal, the level of the internal forces assumed in the dissipative members for the design of the non-dissipative members is increased so that the assumed internal forces of the dissipative members are virtually everywhere equal or greater than the nominal ultimate values. It does not appear that this attempt can be extended to the eccentrically braced structures immediately. In fact, as explained in a previous section, the ratio α_u/α_1 represents the ratio between the load multiplier corresponding to the collapse of the system and the load multiplier corresponding to the formation of the first plastic hinge. Owing to this, the overstrength of the whole system α_u/α_1 also considers the strain hardening of the links and thus cannot be multiplied by $\mathbf{\Omega}_{min}$ (as calculated in the code) because this factor already takes into account the strain hardening of the links.

Even though the use of the amplifying factor $\mathbf{\Omega}_{min}$ may appear unconservative, the Authors are of the opinion that it could be suitable for design for two reasons. First, the resistance of braces, columns and beam segments are generally higher than the minimum values required by the Eq. (5) because sections are selected within the limited number of sections present on the market. Second, in the event of design ground motions the links with high normalised overstrength factor are expected to experience inelastic deformations lower than the ultimate deformation capacity. The use of any amplifying factor of the minimum link overstrength factor would be even less necessary for columns because the seismic axial forces of these members depend on the shear forces of all the links above the storey under examination, i.e., the probability that columns are subjected to high axial forces depends on the probability that the links above the storey under examination are simultaneously subjected to high plastic deformations. Still in regard to Eq. (5), the Authors note (Rossi and Lombardo 2007) that the axial design resistance resulting from its application does not represent a safe evaluation of the axial resistance of braces and columns in the presence of the most unfavourable combination of the axial force, bending moment and shear force. This is because the bending moment and the shear force considered in Eq. (5) are taken at their design value in the seismic situation and, therefore, are far from being equal to the internal forces corresponding to the ultimate plastic rotation of links. The consequences of this erroneous evaluation are mostly evident in the design of systems with intermediate or long links. In fact, in these systems the effect of the bending moments is not negligible when compared with that of the axial forces.

Even more unconservative results are obtained for the beam segments outside links because Eurocode 8 does not require that Eq. (5) should be applied to these parts if horizontal links are used in the structure; in this case, Eurocode 8 only specifies that the shear buckling resistance of the web panel outside of the link must be checked to conform to Eurocode 3. Owing to this, in the majority of the practical situations the design internal forces of the beams are equal to the design value in the seismic situation and, therefore, the safety check is carried out by internal forces which are much lower than the maximum expected values.

In the case of short links, the ultimate internal bending moments at the ends of links are lower than M_p. These bending moments are sustained by beam and brace according to their flexural stiffness and, thus, no yielding of the beam segment outside link occurs. Instead, in the case of intermediate and long links, the bending moment corresponding to the ultimate rotation capacity is higher than M_p (even equal to 1.5 M_p) and the yielding of the beam segment outside the link may occur if no particular solutions are adopted. In this regard, the document AISC 2010 states that it may be useless to increase the size of the beam because the beam and the link are typically the same size. The same document suggests using shear links instead of long links and providing a diagonal brace with a large flexural stiffness so that a larger portion of the link end moment is transferred to the brace. Basically, beams and braces should have sufficient strength to develop the full inelastic strength and deformation capacity of the links (AISC 2010, Engelhardt and Popov 1989a), i.e.

$$M_p^b + M_p^d \geq M_u \tag{31}$$

where M_u is the ultimate bending moment of the link and M_p^b and M_p^d are the plastic bending moments of the beam and brace, respectively. A more conservative approach to this problem is considered in the Italian building code (2008). In this code, eccentrically braced structures are designed according to a procedure similar to that described in Eurocode 8 [Eq. (6)]. However, in the Italian building code not only the axial force but also the bending moment of non-dissipative members are calculated by means of the equations

$$N_{Ed} = N_{Ed,G} + 1.1\gamma_{ov}\,\Omega_{min}\,N_{Ed,E} \tag{32}$$

$$M_{Ed} = M_{Ed,G} + 1.1\gamma_{ov}\,\Omega_{min}\,M_{Ed,E} \tag{33}$$

Table 3 Exemplary case no. 3: sections

Storey	Section			V_u [kN]	M_u [kNm]	$\frac{eV_p}{M_p}$	Ω
	Links	Braces	Columns				
4	HEB 260	HEM 160	HEA 180	324.1	388.9	3.05	1.63
3	HEB 320	HEM 160	HEA 180	541.1	649.3	2.59	1.63
2	HEB 360	HEM 200	HEB 280	669.2	803.0	2.57	1.61
1	HEB 400	HEM 200	HEB 280	795.2	954.3	2.60	1.67

steel grade S235

Table 4 Exemplary case no. 3: internal forces and resisting forces

Storey	$N_{Ed,\,E}$ [kN]	$M_{Ed,\,E}$ [kNm]	$N_{Ed,\,E}$ [kN]	$M_{Ed,\,E}$ [kNm]	$N_{Ed,\,E}$ [kN]	$M_{Ed,\,E}$ [kNm]
	Braces		Columns		Beam segments	
4	358.0	52.3	65.8	6.0	243.5	184.2
3	597.6	52.4	87.9	9.7	400.0	336.6
2	749.2	90.6	380.3	20.6	507.4	400.3
1	855.7	76.6	751.5	29.3	571.6	481.7

Storey	$N_{Ed,\,G}$ [kN]	$M_{Ed,\,G}$ [kNm]	$N_{Ed,\,G}$ [kN]	$M_{Ed,\,G}$ [kNm]	$N_{Ed,\,G}$ [kN]	$M_{Ed,\,G}$ [kNm]
	Braces		Columns		Beam segments	
4	1.3	0.8	158.8	0.1	0.2	3.2
3	5.2	1.8	315.6	0.2	0.9	12.3
2	3.8	1.9	476.7	0.7	0.6	9.2
1	6.7	2.5	634.4	0.5	1.1	15.7

Storey	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed} [kN]	M_{Ed} [kNm]
	Braces		Columns		Beam segments	
4	794. 8	53.2	304.7	6.0	539.8	187.5
3	1329.8	54.2	510.4	9.9	887.5	348.9
2	1664.3	92.5	1319.6	21.3	1125.2	409.5
1	1903.4	79.1	2300.1	29.8	1268.1	497.4

Storey	$N_{b,Rd(M)}$ [kN]	$M_{N,Rd}$ [kNm]	$N_{b,Rd(M)}$ [kN]	$M_{N,Rd}$ [kNm]	$N_{b,Rd(M)}$ [kN]	$M_{N,Rd}$ [kNm]
	Braces		Columns		Beam segments	
4	1705.7	115.7	687.1	62.1	1662.7	274.8
3	1702.4	74.0	666.7	45.3	2193.0	438.9
2	2472.1	137.7	2530.3	233.7	2597.4	530.3
1	2506.2	114.5	2498.6	104.1	2850.1	639.2

The inadequate design of braces and beam segments outside links provided by the provisions stipulated in Eurocode 8 is demonstrated with reference to a four-storey frame with normalised link length e/L equal to 0.30. The geometric and mass properties of this frame are equal at all storeys, as reported in Fig. 2. Again, the P-Δ effects are not taken into account and the vertical load on the links and beam segments outside links is null because of the presence of two beam members at each level of the frame. The system is founded on soil type D and designed by modal response spectrum analysis. The peak ground acceleration is equal to 0.35 g. The cross-sections and steel grade of the members are reported in Table 3. The links are intermediate length at all the storeys except for the top one where the link is long. The internal forces and the resisting forces of braces, columns and beam segments outside links are reported in Table 4. In particular, the symbol $N_{b,Rd(M)}$ represents the buckling resistance reduced due to the bending moment and $M_{N,Rd}$ represents the bending moment resistance reduced because of the axial force. As is evident from this table, in braces and beam segments outside links the bending moment resistance $M_{N,Rd}$ is higher than the bending moment M_{Ed} resulting from the structural analysis. However, the sum of the bending moment resistances in braces and beam segments framing into a single node is not able to balance the ultimate bending moment (1.5 M_p according to Eurocode 8, as shown in Table 3) experienced by the link at the first, second and third storeys. Owing to this, braces and beam segments are doomed to yield or buckle before the ultimate plastic rotation capacity of the links is achieved.

The seismic response of the designed frame has been determined by incremental non-linear dynamic analysis. The seismic input consists of a set of 10 artificially generated accelerograms. The peak ground acceleration is scaled up to the first achievement of the ultimate limit state in the structural components, i.e., to the first achievement of the deformation capacity of links or the inelastic deformation or buckling in non-dissipative elements. In accordance with Eurocode 8-Part 1, the plastic rotation capacity of links is assumed to range from 0.08 to 0.02 rad as a function of the mechanical length. The results show that for all the considered accelerograms the ultimate limit state is achieved because of the yielding of the beam segment outside the link. This limit state is achieved for a peak ground acceleration equal to 0.203 g (value averaged over the 10 accelerograms) and thus lower than the design value. Further, the maximum required plastic deformation in the links is not larger than 60% of the corresponding capacity.

10. Conclusions

This paper summarizes the criticism of researchers on the design provisions for eccentrically braced structures and reports the opinion of the Authors. The objections raised to the design procedure of eccentrically braced structures regard aspects common to the design of steel structures and aspects specific to the design of eccentrically braced structures.

The main conclusions of the paper are:

• The range of application of the lateral force method of analysis for the design of high ductility eccentrically braced structures should be restricted in order to limit the errors in the evaluation of the link overstrength factor.

• The provisions provided in Eurocode 8 to counterbalance P-Δ effects neglect any structural overstrength. Owing to this, these provisions can be particularly conservative for eccentrically braced structures in which the lateral stiffness and strength are increased to satisfy the damage limitation requirement.

• Eurocode 8 evaluates the overstrength factor of links with regard to the ultimate internal forces of these members. This is not in accord with the proposal of Popov *et al*. (1992) and does not ensure a reliable control over the dissipative behaviour of the structure.

• The link overstrength factor is discontinuous at a value of the mechanical length of links and neglects the presence of the gravity loads.

• The design procedure does not seem adequate for structures with intermediate or long links. The rules for the application of the capacity design principles to braces, columns and beam segments outside links are unconservative because of the underestimation of the bending moment

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