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EUROCODE 8 PROVISIONS FOR STEEL AND STEEL-CONCRETE COMPOSITE STRUCTURES: COMMENTS, CRITIQUES, IMPROVEMENT PROPOSALS AND RESEARCH NEEDS

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ABSTRACT

This paper presents a summary of those aspects which are deemed to represent key issues for the correct maintenance of the current Eurocode 8 provisions for the design of steel and composite steel/concrete structures. The general design rules are first commented and, subsequently, aspects specific of each structural type are discussed. Based on the knowledge acquired in the last few years, weaknesses of the current code are then highlighted and improvement proposals presented. Topics requiring further research are eventually identified aiming at paving the way for the next generation of the European seismic code.

KEYWORDS

Seismic design, steel structures, composite steel-concrete structures, moment resisting frames, braced frames.

1 INTRODUCTION

Eurocode 8 (EC8) (CEN 2005*a*) is a prescriptive code, implementing a set of principles and rules to be satisfied in order to meet compliance of the structural performance with the code requirements. Recently, a different idea has encountered the favour of many people in both the scientific and practicing engineering communities: structural codes should be "performance-based" and they should only give the general requirements that a structure should meet. However, the role and value of strictly technical documents, such as the current version of EC8, remains intact, since designers do always need guidance as to how reach general performance objectives.

In the last few years, a strong and passionate discussion has taken place on the subject of the development of rules for seismic design of structures, especially in Italy, where the evolution of the seismic Code has often encountered the resistance of conservative positions. A recent history of this evolution can be found in Landolfo (2005). It seems that a "binary" way of thinking has taken dominance: (i) the "optimistic" view, sustaining the validity of the recent regulations without any doubt against the "old" regulations; (ii) the "pessimistic" view, emphasizing only the "bad" things of the new regulations. According to the Authors' opinion, the time is matured for a more equilibrated approach, which must recognize that the current

version of the Code is a good starting platform, over which modifications for improving the quality and the effectiveness of the new Code can be made.
This paper is a list of shortcomings and/or drawbacks of the current version of EC8 with reference to steel and steel-concrete composite structures. Although it could give the impression that the code is weak, it must be borne in mind that any code has inherent limitations and is susceptible of improvements, as much as the advancement of knowledge proceeds (Landolfo, 2008).
Therefore, the objective of this paper is only to highlight those aspects that, according to the Authors' view, should deserve either further investigation or modification. The discussion presented herein is intended for those specialised people working in the field, since no detailed explanation of the presented issues is given.

2 GENERAL DESIGN CONCEPTS

2.1 Behaviour factors and structural typologies

The force-based design procedure implemented by EC8 relies on the correctness of the behaviour factors. While concerns could be raised about the real background information behind the values of the behaviour factors currently fixed by the code, the lack of information for some important more recent structural types is easily recognized.
Table 1 highlights those aspects which should be either checked or added to the similar table implemented in EC8. Namely, bold characters highlight both those structural types which are currently not dealt with and those aspects which should be checked and eventually adjusted. The following paragraphs discuss very shortly each of the items highlighted in Table 1.
As far as the aspects to be improved are concerned, one controversial issue is about the design of V bracings, where one unique value of the behaviour factor is specified for both ductility class "medium" (DCM) and ductility class "high" (DCH). Indeed, recent research has shown that appropriately designed V-braced frames may reach design values of the behaviour factor of about 4 or even more (Della Corte & Mazzolani 2008). Therefore, the possibility to improve current design rules for V-bracings does exist.
The behaviour factors of eccentric bracings (EBs) is set equal to the values of moment resisting frames (MRFs). Generally speaking, MRFs possess a larger plastic redistribution capacity than EBs. Furthermore, no distinction is made between the use of short, intermediate or long links, even though the plastic deformation capacity of the frame is markedly affected by the type of link.
The mixed reinforced concrete (RC) walls and steel MRF structures are not explicitly dealt with, while they are one attractive solution to designers. Recent studies have shown that the plastic demand to MRFs coupled to RC walls is relatively small (Reyes *et al.* 2009). Consequently, the seismic design of the steel moment frames could significantly be relaxed in this case, meaning that capacity design rules do not need to be fully applied, thus saving the costs. This is one area where further research could profitably be carried out.
Similar to the previous case is the one of MRFs coupled with bracing systems. There are many studies showing that the combination of the two systems may be advantageous, mainly because the large plastic redistribution capacity of the MRFs allows damage concentration in the braced bays to be strongly reduced or even completely avoided.
One recent and very successful application in the field of seismic resistant steel structures is represented by buckling restrained braces. They are not mentioned in EC8 and this is recognized as one main gap in the current version of the code.

Recent research has also proved that one simple but effective way to avoid damage concentration in V-braced frames is to use vertical ties connecting the braces over the frame height ("zipper" bracing). This could also represent one area for the improvement of the code. Finally, the case of MRFs with infills represent one controversial point of the code. In case of "unconnected" MRFs a very small value (q = 2) is assigned to the behaviour factor. This can be interpreted as the result of the brittle response supposed for the (masonry) infill panels. However, the code should recognize that the infill panels usually add stiffness and strength to the frame, thus reducing the displacement demand to the structure. It is frequently found that infilled frames behave better than bare frames. The presence of infill panels should explicitly be considered in the structural model and rules and suggestions should be given as to how this can be done. Besides, the different properties of different infill panels should be taken into account. It is likely that the steel frame is enveloped by a modern cladding system, with larger displacement capacity than classic masonry infills. The case of "connected" infills is correctly thought of as a case of composite structural action, but actually no specific rule is found in the code.

Table 1. Behaviour factors.

STRUCTURAL TYPE	Ductility Class	
	DCM	DCH
a) Moment resisting frames (MRFs)	4	$5\alpha_u/\alpha_y$
b) Concentrically braced frames (CBFs)		
Diagonal bracings	4	4
V-bracings	2 **(?)**	2 **(?)**
c) Eccentrically braced frames (EBFs)	4 **(?)**	$5\alpha_u/\alpha_y$ **(?)**
d) Inverted pendulum	2	$2\alpha_u/\alpha_y$
e) Mixed RC walls and steel MRF structures	**?**	**?**
f) Dual MRFs and CBFs	**4**	$\mathbf{4\alpha_u/\alpha_y}$
g) Dual MRFs and EBFs	**?**	**?**
h) Frames with buckling restrained braces (BRBs)	**?**	**?**
i) Frames with metallic shear panels (SPs)	**?**	**?**
j) "Zipper" bracing	**?**	**?**
k) MRFs with infills		
Unconnected	2 **(?)**	2 **(?)**
Connected	See Section 7 **(?)**	
Isolated	4	$5\alpha_u/\alpha_y$

There is a number of novel structural types which are spreading all over the World and should be included in the next generation of EC8.
The following is a list of such novel types:

1. Frames with buckling restrained braces.
2. Frames with metallic plate shear walls.
3. Special braced frames, with improved performance, such as the "zipper" bracing.

Many theoretical and numerical studies have recently been performed on these structural types (D'Aniello *et al.* 2008, De Matteis *et al.* 2007, Yang *et al.* 2007), which could be considered mature enough to be included in the code. It is worth noting that the first two types of novel systems is already included in the current AISC (2005) Seismic Provisions.

2.2 Classification of cross sections

Steel member cross-sections are classified by EC8 according to the same rules fixed by Eurocode 3 (EC3) (CEN 2005*b*). It has long been recognized that the classification for monotonic loading must be different from the one for seismic loading, because of strength deterioration induced by the repetition of inelastic deformations. Furthermore, it has been pointed out that shifting from a section-based to a member-based classification would represent one significant advancement of the code. More discussion about the cross section classification is provided at Section 3.1.

2.3 Material random overstrength

A well-established general concept in seismic design of structures is that non dissipative members must be designed on the basis of the expected material strength of the dissipative zones. The ratio between the expected (average) yield stress and the nominal yield value for a given steel class is called γ_{ov} by EC8. There is no specific information about the values to be attributed to γ_{ov}, but National Authorities have the freedom to select the most appropriate ones. However, a constant value of $\gamma_{ov} = 1.25$ is suggested, which is contradictory with the available experimental evidence of the dependence on the yield strength of the steel (Calderoni *et al.* 1994).

2.4 Capacity design

The rules implemented for capacity design in case of steel structures are different from those implemented for other materials, and this deserves some comments.

In case of steel structures, capacity design of non dissipative parts is regulated by a unique format applicable to all the different structural types covered by the code. Namely, earthquake-induced effects are increased by the factor $1.1\gamma_{ov}\Omega$, where γ_{ov} has previously been defined and $\Omega = \min\left(R_{pl,Rd,i} / R_{Ed,i}\right)$, where $R_{pl,Rd,i}$ is the design strength of the *i*-th plastic zone and $R_{Ed,i}$ is the required strength. Therefore, the design value of the generic internal action for non dissipative members is taken equal to $R_{Ed,i} = R_{Ed,G,i} + 1.1\gamma_{ov}\Omega R_{Ed,E,i}$, where subscripts "*G*" and "*E*" indicates the effect of gravity and earthquake loads, respectively. There are some controversial aspects in this approach. In case of small gravity load effects, amplifying the earthquake-induced counterpart of the required strength by the factor $\gamma_{ov}\Omega$ means that the internal actions corresponding to the first real plastic hinge formation are being calculated. However, in case of large gravity load effects, the proposed Ω factor markedly underestimates the real overstrength. One proposal of correction has been recently reported by Elghazouli (2008) and was formerly proposed within the first version of the recent Italian Seismic Code (OPCM 3274). The meaning of the multiplicative coefficient (1.1) is not clearly stated in the code. According to Elghazouli (2008) it is introduced to take into account strain hardening of steel and strain rate effects. Indeed, the coefficient 1.1 is proposed by the code also for capacity design of connections between a plastic zone and non dissipative parts of the structure. In this case the expected yield strength of the plastic zone $\gamma_{ov}R_{yd}$ is amplified by a factor again equal to 1.1. Alternatively, one could suppose that, in case of capacity design of columns, the coefficient tries to take account of the force redistribution occurring after the first plastic hinge formation. But, in this case using one single coefficient for any type of structure and any type of design criteria would be strongly questionable. In fact, the force redistribution capacity of MRFs is markedly different from that of braced frames and, for a given structure type, it may significantly change according to the design criteria used. Therefore, the exact meaning of this coefficient (1.1) and, especially, the rational background behind the assumed value, remains unknown. Rather, it seems to be one of those "magic" numbers which are sometimes encountered in the code, when a clear scientific background is missing. Amplifying the action effects due to earthquake loads only is also questionable, since the ratio between the actual strength of plastic zones and the strength required by all the loads should affect the internal actions to design non dissipative members and connections.

2.5 Structural regularity

The code suggests reducing by 20% the behaviour factor of buildings which are irregular in elevation. This rough estimation of the effect of vertical irregularity seems to be an oversimplification, which should deserve a deeper investigation. Many numerical results are available on this subject, which also shows that geometrical set-backs could even be beneficial in some cases (Mazzolani and Piluso 1997).

2.6 Floor diaphragms

Design rules for floor diaphragms and their connections with the vertical frames are lacking within the current version of EC8. Indeed, the code suggest multiplying the effects obtained from the seismic analysis by an overstrength factor γ_d, whose value is to be found in the National Annex. But, constant values equal to 1.3 and 1.1 are suggested for brittle and ductile failure modes, respectively. This assumption of a unique value, independent on the type of structure and its design criteria, seems to be inadequately oversimplified and should deserve further investigation.

2.7 Foundation connections

An harmonization of the design of foundation connections with the capacity design of columns is required. In particular, in case of composite steel-concrete constructions, the large flexural strength of composite columns poses serious problems to the practical implementation of capacity design rules (Di Sarno *et al.* 2007).

3 DESIGN ASPECTS SPECIFIC OF EACH STRUCTURAL TYPE

3.1 Moment resisting frames

Seismic design criteria for MRFs have long been studied (Mazzolani and Piluso 1995), but many results still needs to be fully exploited in order to justify the assumptions for the q-factor values in relation to the design criteria.

A very general problem with the current codified EC8 rules for the design of MRFs is the compatibility of maximum drifts imposed at the damage limitation limit state and the large behaviour factors for the ultimate limit state (Della Corte *et al.* 2002). It has long been recognized that the design of MRFs is dictated by drift limitations of EC8, which produces strong overstrength and, consequently, reduced ductility demand as well as increase of costs. Limitations on P-Delta effects could also be a source of significant frame overstrength (Elghazouli 2008).

Another important weakness of the current version of EC8 can be identified in the lack of any regulation about the detailing of moment resisting connections. There is a large amount of experimental and theoretical information about the response of beam-to-column connections

and panel zones, which have been basically produced in response to the Northridge and Kobe earthquakes (Mazzolani 2000, FEMA 2000). This information could profitably be used to form the basis of an upgraded code.
Analogously, one more area where the code could easily be improved, by fully exploiting the existing knowledge, is the classification of member cross sections. One useful proposal is that made by Mazzolani and Piluso (1996), who propose to establish a relationship between the member ductility and the stress ratio $s = f_c / f_y$ (i.e. the ratio between the peak collapse stress f_c and the yield stress f_y). Through a regression analysis of many experimental test results on beams with I-shaped cross-sections, the stress ratio is obtained as function of the flange and web slenderness (λ_f and λ_w), as well as the ratio between the distance of the cross section subjected to the peak bending moment from the contraflexure cross section (L^*) and the beam flange width (b_f). Recently, an upgrading and extension to tubular member cross sections has been made (Landolfo *et al.* 2008, Brescia *et al.* 2009). Based on the parameter *s*, the proposed cross section classification is given in Table 2. One additional advantage of the proposed classification is that only 3 classes are defined. Indeed, the difference between class 2 and class 3 of EC3 is somewhat troublesome from a conceptual point of view and usually a very small number of cross sections belong to class 2. Besides, the four classes based classification does not find any correspondence in other international codes.

Table 2. Classification of cross sections.

Cross section class	Cross section normalized strength (*s*)
Ductile	$s \geq 1.2$
Plastic	$1 \leq s \leq 1.2$
Slender	$s \leq 1$

The possibility of using slender cross sections, such as those typically characterizing cold-formed members, along with a *q*-factor larger than 1 has long been investigated by Calderoni *et al.* (2008). The current codification strongly penalizes the use of cold-formed members, since only a non-dissipative structural design approach is permitted for them.
The case of composite construction is one field of application where more research is needed with reference to MR frames. For example, the use of partially restrained vs. fully restrained moment connections should be clearly distinguished by the Code and specific design rules formulated for each of the two options. Significant experimental and theoretical studies have recently been carried out and some useful results are available (Thermou *et al.* 2002, Amadio *et al.* 2008, Bursi *et al.* 2008). As a further example, the currently codified rules for the classification of cross sections appear to be an oversimplification of the real problem. Recent research results (Pecce *et al.* 2009) could form the basis for an improvement of the code, by looking at consolidated knowledge for bare steel frames (Mazzolani and Piluso 1996).

3.2 Concentrically braced frames

The design of CBFs is regulated according to somewhat different criteria depending on the type of bracing. In fact, in case of diagonal bracing, the use of an elastic model with only-tension braces is prescribed, while in case of V bracings both the tension and compression braces shall be taken into account.
While using one single brace to calculate the ultimate storey shear strength could be considered as an acceptable simplification for slender braces, the adoption of an elastic model

with one single brace is difficult to justify even in this case. The procedure overestimate the elastic period of vibration, hence underestimate the elastic force demand on the braced frame. Consequently, a premature and uncontrolled compression buckling of braces will occur.

The post-buckling strength of braces is required in the design of beams in V bracings. The code assumes this post-buckling strength to be a fraction of the plastic strength in tension, i.e. $\gamma_{pb} N_{pl,Rd}$. Though the post-buckling strength coefficient γ_{pb} can be selected by each National Annex, a constant value equal to 0.3 is suggested. But, there is an extensive literature showing that the post-buckling strength is function of the brace slenderness (Tremblay 2002).

There is missing information in the code about the design of some types of unfrequent, but usable, bracing systems, such as X-bracing systems over two or more floors and bracing with vertical ties ("zipper" frames).

One point of the code deserving particular attention is the requirement about the height wise variation of the overstrength factor Ω_i. The difference between the values of Ω_i at two adjacent floors is prescribed to be not larger than 25%. The objective is clearly to avoid or, at least, to limit the occurrence of damage concentration in one or few stories. However, the coupling of this requirement with the upper limit on the brace slenderness $\overline{\lambda}_{br,i} \leq 1.3$, which is imposed in case of V-bracing, may produce strongly over-resistant braces. This occurs because the seismic shear force at the top storey is usually small, thus leading to small brace strength demand. Consequently, it would often be appropriate to select small cross section area and radius of inertia, i.e. large slenderness, at the top floor. In order to satisfy the limit on the brace slenderness, the designer is then forced to select larger cross sections at the top floor and, consequently, to every floor, because of the limitation on the brace design overstrength factor. Once braces have been overdesigned, the application of the codified capacity design rules obviously leads to large cross section areas for columns and beams, as well as to large force demand on connections. This design solution is obviously expensive, though the final overstrength will be on the safe side. One very simple way to avoid the problem, could be assigning at the top storey a conventional additional force, as percentage of the base shear force. This could also be advantageous to face higher mode effects which are pronounced at the top storeys.

Information is missing about using double-angle or double-channel cross section shapes for braces. For example, information about the stitch spacing is not provided, while experimental tests have proved that the brace ductility is significantly affected (Astaneh-Asl *et al.* 1996). Double angle and double-channel shapes are frequently encountered in the European Countries, where low-to-moderate seismic intensities are frequent.

Information is also missing about the design and detailing of brace to beam and column connections. The sensitivity of the real brace performance to the detailing of the connections is well known and documented by several experimental studies (Astaneh-Asl *et al.* 1998). Braces buckle by forming three plastic hinges, two at the ends and one at the middle of the brace. In case of out-of-plane buckling, the plastic hinges at the brace ends could be either permitted to form in the gusset plate connections or forced to occur in the brace. Though the latter option is characterized by larger energy dissipation capacity (Lee and Goel 1987), the former solution is economically advantageous, because it avoids using gusset plate transverse stiffeners. If plastic hinges are permitted to form in the gusset plates, then an appropriate length of the free space between the end of the brace and the assumed line of restraint for the gusset must be detailed (AISC 2005, Astaneh-Asl *et al.* 1986) in order to accommodate a plastic hinge but avoiding buckling . Besides, recent research has shown that alternative brace end connections could advantageously be employed (Martinez-Saucedo *et al.* 2008).

3.3 Eccentrically braced frames

According to EC8, capacity design of non dissipative members and connections outside the yielding link zone shall be conducted using the general rule of Section 2.4. The product of the material overstrength factor γ_{ov} and the Ω factor is shown in Figure 1*a*, as function of the normalized link length. An unacceptable and inconsistent discontinuity is observed at $\rho = 1.6$, i.e. at the transition between short and intermediate link lengths ($\rho = V_p e / M_p$ is the normalized link length). The EC8 codification is compared in Figure 1*a* with the one reported by Richards and Uang (2002). The inherent link overstrength of European shapes is considered to be larger than for US shapes. This is consistent with some experimental findings (Mazzolani *et al.* 2009). However, theoretical studies also suggest that the link overstrength should be considered length-dependent and shape-dependent (Della Corte *et al.* 2009). This is not currently recognized by the codes and could indeed represent one area of further research. Besides, the link classification is proposed for only I-shaped cross sections, what implies that the use of different cross section shapes is actually not possible. However, some documentation is available for other types of cross section, such as the tubular one (Fig. 1*b*). The available experimental and theoretical studies could form the basis for an extension of the design rules, making possible for the designer to select the preferred shape of cross section. This could be particularly important in some European Countries, such as Italy, where tubular shapes are easily available in the market as cold-formed products.

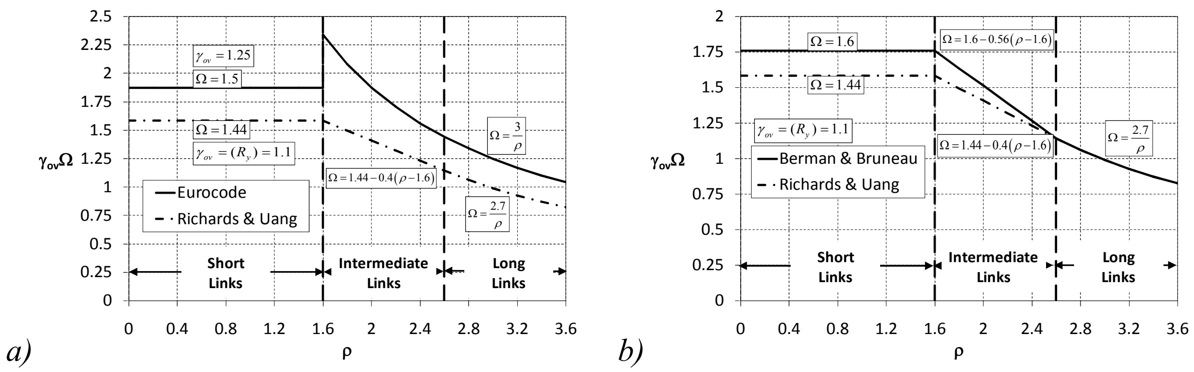

Figure 1. Link classification and overstrength.

3.4 Dual systems

Information on dual structural types (e.g. reinforced concrete walls and moment resisting frames, moment resisting frames with bracing) is lacking in the current version of EC8. Research results are available for such dual systems (Dubina *et al.* 2008, Reyes *et al.* 2009), which are among the most efficient structural types worth of consideration.

4 CONCLUSIONS

The paper has shortly commented a few issues related to the current Eurocode 8 regulations for the design of steel and composite steel concrete structures, highlighting those aspects deserving improvement or further research to be carried out. The Authors are aware that many other issues could/should be discussed and that the presented list is not exhaustive. However, the short list presented here could form the basis to plan the maintenance operations of the current version of the code. It is worth mentioning that such a maintenance process has

already started within the ECCS TC13 Committee about the seismic design of steel and composite steel-concrete structures (www.steelconstruct.com).
A "good" Code must necessarily have two requisites: (i) it must be equipped with a Commentary, explaining the reasons for the prescribed rules and (ii) the scientific and technical background of the Code must clearly be described (e.g. in the same Commentary). Unfortunately, both of these two requisites are not satisfied by the current version of EC8 and many of the rules prescribed in the Code are consequently obscure even to people working in this field either as researchers or practitioners. Notwithstanding, Eurocode 8 represents an advancement in the field of seismic design for European structures, with strong efforts done by many people involved into the Code development. The current version should profitably be used as the starting basis to develop more comprehensive and clear design rules.

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