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Seismic design criteria for chevron CBFs: Proposals for the next EC8 (part-2)

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ABSTRACT

According to EN 1998-1, chevron concentrically braced frames (C-CBFs) are expected to provide limited ductility due to the interactions between the braces and intercepted beam. As a consequence, EN 1998-1 recommends values of the behaviour factor (namely $q = 2$ and $q = 2.5$ in ductility class medium and high, respectively) lower than that (i.e. $q = 4$) given for other concentric bracing configurations, e.g. both cross and single diagonal CBFs. The research activity presented in this paper is addressed to revise the design rules and requirements of EN 1998-1 for C-CBFs in order to improve the ductility and the dissipative capacity of this structural system. The study is organized in two parts: in the first part design rules recently presented in literature are assessed, while in the second part new design criteria are proposed and validated against the results obtained from incremental dynamic analyses carried out on 2D frames extracted from low, medium and high rise buildings.

1. Introduction

Chevron concentrically braced frames (C-CBFs), also known as inverted-V CBFs, are very popular as primary seismic resisting system for steel multi-storey frames. However, it is well known that this structural system is less ductile than other braced configurations. Indeed, under seismic loading C-CBFs may suffer significant degradation of both strength and stiffness due to concentration of large deformations into the braces in post-buckling range, which causes severe reversed cyclic rotation at the plastic hinges formed into the braces with also high bending demand on the beam at brace intersection [1–4].

Due to these features, according to EN 1998-1 (hereinafter indicated also as either Eurocode 8 or EC8) [5] C-CBFs are expected to provide relatively limited ductility. Therefore, EC8 recommends behaviour factors (i.e. q equal to 2 for ductility class medium "DCM" and 2.5 for ductility class high "DCH") smaller than those given for X-CBFs and diagonal bracings (i.e. q equal to 4 for both medium and high ductility class). It is interesting to note that EN 1998 [5] recommends different behaviour factors per type of CBF configuration. On the contrary, North American codes assign the response modification factors and the ductility class regardless the bracings configuration. Even though both European [5] and North American [6,7] seismic codes adopt the capacity design philosophy, the design rules and requirements for both dissipative and non-dissipative zones are different, thus affecting the global response and the energy dissipation capacity of the system as deeply investigated in the companion paper [8], where the main criticisms of EN 1998-1 [5] are clearly identified. In particular, one of the main criticism about the design rules for dissipative members is the requirement on the variation of capacity-to-demand ratio of bracing members or overstrength factor $\Omega_i = N_{pl,br,Rd.,i}/N_{Ed,br,i}$ that should vary within the range Ω to $1.25\,\Omega$, where $\Omega = \min(\Omega_i)$, $N_{pl,br,Rd.,i}$ is the plastic axial strength of bracing members at the i-th storey and $N_{Ed,br,i}$ is the relevant seismic demand. The recent studies carried out by [8,9] clearly show that this rule (which is not included in North American seismic codes) does not ensure uniform distribution of plastic deformations along the building height. With reference to non-dissipative members, EC8 [5] allows using simplified procedure for calculating the forces acting in the columns belonging to the braced bays. Indeed, plastic mechanism analysis is not required for columns and it is sufficient to perform only an elastic analysis without specifically accounting for the distribution of internal actions occurring in the post-buckling range. This calculation method gives less conservative estimation of internal forces than those calculated according to both US and Canadian codes [6,7], thus largely underestimating the earthquake-induced effects into the columns in the most of cases. Another deficiency of EC8 concerns the design of the brace-intercepted beam. Indeed, the rules prescribed by EN 1998 lead to design weaker beams as respect to AISC 341 [6] and CSA-S16 [7]. In light of the findings shown by [8–13], which highlighted that the flexural stiffness of the brace-intercepted beams significantly affects the seismic performance of C-CBFs, additional requirements devoted to control the beam flexural stiffness, beside its strength, are necessary in order to avoid the disproportionate deterioration of braces response under compression and to allow full yielding of diagonals under tension, as well.

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Several authors [14–21] proposed design criteria devoted both to overcome the fallacies of EC8 and to improve the seismic performance of CBFs, by accurately accounting for the nonlinear behaviour of diagonal members under cyclic conditions.

Marino [14] proposed a unified approach for the seismic design of high ductility steel frames equipped with concentric bracings (whatever configuration is considered). The method is based on design criteria previously developed by [15] for chevron bracings. According to this procedure, a behaviour factor equal to 3.5 (i.e. larger than 2.5 that is the EC8 recommended value for high ductility structures) is assumed and the storey lateral resistance is evaluated assuming that the tension and compression bracings attain their full yielding and post-buckling strength, respectively.

Remarkable efforts in the improvement of seismic performance of chevron bracings can be found in [18–21], where it was clearly highlighted that the design recommendations provided by EN 1998-1 [5] are not adequate to ensure ductile global failure mode and to avoid soft-storey mechanisms. In order to overcome the limits of EC8, Longo et al. [19–20] and Giugliano et al. [21] proposed a new design methodology to guarantee the development of a global collapse mechanism. With this aim, the axial forces acting in non-dissipative members are evaluated considering that the braces are yielded at each storey. This approach overcomes the criticism of the EC8 concerning the underestimation of axial force acting into the columns in the braced span. On the other hand, this design approach leads to an increase of the structural weight (and consequently larger constructional costs), without reaching significant benefits in terms of yielding in tension and energy dissipation capacity. Such a result can be explained considering that the deformed configuration at failure assumed by [19–21] does not account for the contribution due to the vertical deflection of the braced-intercepted beams, while the findings obtained by [12,13] demonstrated that the flexural deformation of the beams significantly influences the ductility demand of both tension and compression braces.

As confirmed by existing literature, the field of codification review is currently prolific in Europe [22,23]. With this regard it is even worth mentioning that an European six-year work program is currently ongoing to amend and revise all Eurocodes, including also new findings and the advances of knowledge coming from research [23].

In line with this purpose, the current EC8 rules are revised and new design criteria for ductile chevron concentrically braced frames are also proposed in order to enhance the seismic performance and energy dissipation capacity of C-CBFs. The effectiveness of proposed design requirements is validated by means of parametric nonlinear dynamic analyses carried out on low, medium and high rise buildings.

This paper is organized into two main parts, as follows: i) Section 2 deeply investigates the influence of the EC8 requirement on the capacity-to-demand ratio (namely the overstrength factor) variation, analysing the validity of two potential revisions of the current EC8 formerly discussed within [22,23]; ii) in Section 3 new seismic design criteria for ductile chevron concentrically braced frames for the next generation of Eurocodes are described and discussed by means of results of non-linear dynamic analyses.

2. Criticisms on the requirement in brace ovestrength variation

2.1. Current state

Concentrically braced frames are low redundant structures that are typically prone to soft-storey mechanisms. In order to limit this detrimental behaviour and to improve the number of storeys involved in the plastic mechanism, EN 1998-1 [5] mandates to limit the variation of the brace overstrength ratio Ω_i along the building height. As briefly discussed in the previous Section, a large number of numerical studies [8,12–21,24] showed that soft-storey mechanisms cannot be avoided, even restraining the storey-to-storey variation of Ω_i within the limits mandated by EC8. Indeed, EC8 compliant C-CBFs generally exhibit a cantilever-type displacement shape, with severe damage concentration at upper storeys and bracings under tension behaving in the elastic range up to near collapse limit state [8,9].

In addition, to meet such requirement entails significant efforts and practical difficulties in the design process, because in the most of cases the diagonal members at upper storeys should be oversized to comply with the upper bound limit of normalized slenderness, namely $\bar{\lambda} \leq 2$. As a consequence, the braces at lower storeys must be overdesigned in order to fulfill the requirement on overstrength variation, thus often leading to impractical and expensive solutions.

Since the typical values of overstrength Ω range around 2, very limited plastic engagement of the bracings under tension can be exploited [24–26]. In addition, the sequence of brace buckling along the building height is not uniform, because limiting solely the variation of brace overstrength in tension (namely $\Omega < \Omega_i = N_{pl,br,Rd.,i}/N_{Ed,br,i} < 1.25\,\Omega$) entails disregarding the storey-to-storey variability of brace overstrength in compression.

This feature is detrimental because storey strength and stiffness abruptly drops after brace buckling with subsequent damage concentration [8,12,14,15,27].

2.2. Potential upgrading

In order to mitigate the undesired effects above described and to design effectively the braces along the building height, two alternative revisions have been formerly discussed within the activity for the amendment of Chapter 6 of EN 1998-1 for the new version of Eurocodes recently carried out by TC13 (i.e. Technical Committee 13 – seismic design) of the European Convention for Constructional Steelwork (ECCS) [22,23]. These potential improvements are described as follows:

(i) The requirement on Ω_i variation is retained, but excluding the braces at roof storey. Therefore, considering an *n*-storey building, it is imposed that:

$$[(\Omega_i - \Omega)/\Omega] \leq 0.25 \tag{1}$$

where $\Omega = \min(\Omega_i)$ and Ω_i is the overstrength ratio at the *i*-th storey evaluated as:

$$\Omega_i = \frac{N_{pl,br,Rd,i}}{N_{Ed,br,i}} i \in [1, (n-1)]. \tag{2}$$

This adjusted rule (hereinafter referred as "Ω_1 criterion") basically aims at avoiding to overdesign the diagonals at lower and intermediate storeys, excluding the braces at the roof storey that are generally characterized by large overstrength if the EC8 verifications on both the strength and slenderness should be satisfied.

(ii) The requirement on Ω_i variation is retained including the roof storey, but the overstrength ratio at each storey is defined considering the compression axial strength of the brace $N_{b,br,Rd.,i}$ rather than the tensile plastic strength, being the buckling of the brace under compression the first nonlinear event occurring at each storey. In light of this consideration, the brace overstrength is defined as follows:

$$\Omega_i = \frac{N_{b,br,Rd,i}}{N_{Ed,br,i}} = \frac{\chi_i N_{pl,br,Rd,i}}{N_{Ed,br,i}} \tag{3}$$

It is interesting to note that this rule (referred in the following as "Ω_2 criterion") makes easier to fit the distribution of diagonal strengths $N_{b,br,Rd.,i}$ to the distribution of computed action effect $N_{Ed,br,i}$, thus theoretically enforcing a uniform sequence of brace buckling that might be beneficial to limit the activation of soft storey mechanism.

2.3. Analysis and discussion of results

In order to assess the effectiveness of these revised criteria, non-