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# Assessment of the Design Provisions for Steel Concentric X Bracing Frames with Reference to Italian and European Codes

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**Abstract:** In the field of construction in seismic areas, the current Italian technical code for constructions NTC2008 is substantially based on the design criteria of Eurocode 8 (EC8), although with some differences. Focusing on steel structures with X Concentric Braces (CB), which is one of the most common seismic resistant structural typology in steel buildings, the paper illustrates a critical review of design methodologies specified in NTC2008, with the intent of providing simplified and more efficient design criteria and procedures able to ensure adequate safety levels under seismic actions, according to the modern design approach. The study is divided in two parts. The first part consists in the design according to the standard rules of typical steel X braced structures by linear analysis, both static and dynamic. The aim is identifying any possible weakness in the current design criteria, with particular reference to both the applicability of the proposed procedures and the actual possibility to size the bracing cross-sections and the connected structural members, like beams and columns. The second part consists in the assessment of the seismic response of structures examined. To this purpose, non-linear static analyses are performed in order to evaluate the most relevant behavioural issues, like the behaviour factor, the failure modes and the effectiveness of the capacity design criteria. Based on the results obtained, a proposal for the enhancement of design criteria is presented.

**Keywords:** Behaviour factor, concentrically braced frames, non linear static analyses, overstrength factor, seismic design criteria, seismic resistant steel structures.

## 1. THE RESEARCH CONTEXT

In recent years a twofold occurrence has motivated the review for maintenance of technical codes for design and construction in seismic areas: on one hand the huge amount of research results and advances in the state of knowledge in the field of seismic engineering, on the other hand the frequent recurrence all over the world of seismic events of high intensity and serious consequences. Unfortunately this is the actual context also for Italy, which has been recently theatre of severe earthquakes striking both historical centres and modern buildings, even devoted to productive activities. With particular focus on steel structures, a crucial moment for the development of seismic design codes for steel constructions has been the draft of OPCM 3274 and 3431 [1] since 2003, which introduced an extensive chapter, in line with EC8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings [2], with the addition of some noticeable changes on the design of steel structures with respect to the European standard, integrating the evidences of extensive studies on the seismic behaviour. However these amendments were not included in the current technical standards, in a view of guarding the symmetry with EC8 [3]. What is more, the current Italian NTC2008 [4] has several cuts with respect to EC8, which were partially recovered in the explicative Italian Ministerial Circular [5].

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Therefore the need to fill this gap, incorporating and merging all the most updated achievements of research related to the design of seismic resistant systems motivated the Italian project RELUIS-DPC (2010-2013), specifically focused on these issues. In particular, the task of the research unit UNINA-ING was the optimization of design criteria for seismic resistant steel braced structures. First results of this research activity were provided in De Lucia *et al.* [6] and Macillo *et al.* [7, 8], with respective reference to X-braced structures and chevron braced structures, the latter being described in Castaldo *et al.* [9]. The research is ongoing in the framework of the new edition for the year 2014 of the Italian project RELUIS-DPC.

## 2. NTC2008 DESIGN CRITERIA FOR CBF-X STRUCTURES

In Concentrically Braced Frames (CBF), the resistance against the seismic actions is provided by the contribution of both tensile and compression braces. The ideal design ultimate condition of a dissipative braced system is the simultaneous buckling of compression bracings and yielding of tensile ones, the braces being the dissipative elements [10, 11]. According to the commonly accepted resistance hierarchy criterion, the other structural elements, such as columns, beams and connections, have to remain in elastic range and, therefore, they should be designed to have an adequate overstrength as respect to braces. Hereafter the Ultimate Limit State (ULS) design rules for seismic resistant systems with X braces (CBF-X) are summarised.

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The results of the design phases in terms of member profiles of the different investigated structures together with the total weight of each member type are provided in Tables from **1** to **3**.

## 4. DESIGN ASSESSMENT OF CBF-X

The NTC08 design criteria show a first critical issue in the ambiguity of the design procedure in the use of linear dynamic analysis results. In case of CBF-X, the code prescribes that at the ultimate limit state only the braces in tension resist the seismic forces, while the compressed braces are considered buckled and unable to provide strength. Nevertheless the vibration properties of the structure, i.e. periods and vibration modes, are strictly related to the linear behaviour and they should be determined considering the contribution of both braces in tension and in compression, therefore they cannot be calculated disregarding braces in compression. For this reason, for the structures examined the linear dynamic analysis is performed by considering the presence of both braces not only for evaluating the elastic vibration properties, but also for assessing seismic forces in the members. Then, in order to consider the model with only one active diagonal, the design of braces is carried out by assuming a value of the axial forces as the double of the one calculated by means of the structural model including both tensile and compressed diagonals (Fig. **2**). Based on this, a possible improvement of the design criteria for CB-X could be to clearly state this procedure within the code.

The design results show that the CBF-X structures designed with linear static (LS) analyses are generally subjected to seismic actions higher than those designed through linear dynamic (LD) analyses, as it is apparent from Table **4**, where, with reference to the single CBF-X, W is the structural weight, T is the fundamental period of vibration, $F_h$ is the design base shear and $\Omega_{min}$ is the design overstrength factor.

This difference is mainly related to the underestimation of the fundamental vibration period through the empirical formula provided by NTC2008 in case of linear static analyses. This issue is more evident for taller buildings. For instance, in the case of 10-storey structures, the vibration period calculated by the code formula is 42% smaller than the one evaluated through dynamic analysis, with a consequent 61% increment of the total seismic force. This issue also influences the weight of the seismic resistant members. In particular, the structural weight of the structures designed through static analysis are up to 25% higher than those obtained by dynamic analysis. Based on this result, a possible improvement of the design criteria for CB-X could be to define different simplified relationships for the preliminary determination of the fundamental period of vibration, depending on the number of floors.

Another critical issue observed in the design phase is the difficulty in selecting the bracing profiles. In particular, the lower bound of $\bar{\lambda}$ (equal to 1.3) strongly limits the HE profiles that can be used. In addition, the low seismic demand at upper storeys implies oversized bracings with corresponding very high $\Omega$ values. This especially occurs at the top storey, where the condition of uniformity of the $\Omega$ factor distribution along the structure height is hard to be satisfied (eq.3). Thus, for the 10-storey structures examined the top storey has not been considered in the check of the requirement concerning $\Omega$ values. Based on this results, a possible improvement of the design criteria for CB-X could be to define more pertinent rules for the top storey. Some authors proposed a different approach based on the reduction of the bracing members section at the ends to obtain $\Omega$=1 [13, 14]

## 5. NUMERICAL MODELLING ISSUES FOR CBF-X

Structural analyses are performed by means of the FEM software SAP2000 v. 14.0.0 [15]. Members are modelled as beam elements with lumped plasticity, columns are continuous along the total height and both beam-to-column and

**Table 1. Member profiles for 10 storeys CBF-X structures.**

| Storey | LS | | | LD | | |
|---|---|---|---|---|---|---|
| | Diagonal | Column | Beam | Diagonal | Column | Beam |
| 10 | HE 100 A | HE 180 B | IPE 220 | HE 100 A | HE 180 B | IPE 220 |
| 9 | HE 100 A | HE 180 B | IPE 220 | HE 100 A | HE 180 B | IPE 220 |
| 8 | HE 120 A | HE 260 B | IPE 270 | HE 100 A | HE 240 B | IPE 220 |
| 7 | HE 140 A | HE 260 B | IPE 270 | HE 120 A | HE 240 B | IPE 240 |
| 6 | HE 140 B | HE 360 B | IPE 300 | HE 120 A | HE 280 B | IPE 240 |
| 5 | HE 140 B | HE 360 B | IPE 300 | HE 120 B | HE 280 B | IPE 270 |
| 4 | HE 140 B | HE 360 M | IPE 330 | HE 120 B | HE 280 M | IPE 270 |
| 3 | HE 100 M | HE 360 M | IPE 330 | HE 120 B | HE 280 M | IPE 270 |
| 2 | HE 100 M | HE 500 M | IPE 330 | HE 140 B | HE 300 M | IPE 300 |
| 1 | HE 100 M | HE 500 M | IPE 330 | HE 140 B | HE 300 M | IPE 300 |
| Member weight [kN] | 43 | 115 | 28 | 33 | 96 | 20 |

## CONFLICT OF INTEREST

The authors confirm that this article content has no conflict of interest.

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