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APPRAISAL OF SEISMIC DESIGN CRITERIA FOR CONCENTRIC BRACING STEEL STRUCTURES ACCORDING TO ITALIAN AND EUROPEAN CODES

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SUMMARY: *The critical review of design methods provided by the Italian codes for constructions NTC2008, in agreement with the European seismic code (EC8), for steel Concentrically Braced Frames (CBF) with X and chevron (or inverted V) diagonals, has the aim at providing more efficient design criteria able to ensure adequate safety levels under seism. As reference case studies, common structural configurations of CBF are designed according to NTC2008, by both Linear Static and Dynamic analyses. The purpose is identifying the weaknesses in the design criteria, with particular reference to both the applicability of the proposed procedures and the actual possibility to size braces and connected structural members, like beams and columns. The critical issues in the design process are evidenced. A discussion on the obtained results has allowed to point out the pros and cons of the current design approach and to propose some enhanced design criteria.*

KEYWORDS: *Seismic resistant steel structures, Seismic design criteria, Concentrically braced frames, Behaviour factor, Over strength factor, Non linear static and dynamic analyses*

1. THE RESEARCH CONTEXT

In recent years the huge amount of research and advances in the field of seismic engineering, together with the frequent occurrence worldwide of seismic events of high intensity and serious consequences, not last the case of Italy, recently stroke by severe earthquakes, has motivated the upgrading of technical codes for constructions. With particular focus on steel structures, a crucial moment for the development of the national seismic design codes has been the draft of OPCM 3274 and 3431 [2005] since 2003, which introduced an extensive chapter, in line with EC8: Design of structures for earthquake resistance [EN 1998-1, 2005], with the addition of some noticeable changes 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, for the sake of symmetry with EC8 [Mazzolani and Della Corte, 2008]. What is more, the current Italian NTC2008 [M.D., 2008] has several cuts with respect to EC8, which were partially recovered in the explicative Italian Ministerial Circular [M. C., 2009]. A general overview of the design aspects and applications related to steel CBF structures, among the other steel seismic resistant structures, is presented in Mazzolani *et al.* [2006], where the modern approach is discussed, evidencing the conformity between the Italian and European

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codes. A more rigorous plastic design procedure for CBFs was proposed by Longo et al. [Longo *et al.*, 2008a; 2008b], more recently Marino has proposed a unified design approach for CBFs [Marino, 2014]. A wide research activity, aiming at the progressive upgrading and optimization of seismic design rule for constructions, including steel braced structures, is ongoing within the Italian project RELUIS-DPC. In this context, a deep analysis of X and V braced structures has been presented in Faggiano *et al.* [2014; 2015a,b] and in D'Aniello *et al.* [2013, 2015].

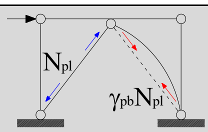
2. NTC2008 DESIGN CRITERIA FOR CBF STRUCTURES

2.1 General aspects

In Concentrically Braced Frames (CBF), the resistance against 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. 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 over strength as respect to braces. In Table 1 the Ultimate Limit State (ULS) design rules for seismic resistant concentric braced systems are summarised.

Firstly, members should be ductile, thus belonging to Class sections 1 or 2, according to the cross section classification defined in Eurocode 3 [EN 1993-1-1, 2005; Formisano *et al.*, 2006] and taken by NTC2008. In order to prevent the untimely collapse of beams and columns, according to the hierarchy design criterion, the code requires to determine the overstrength factor *Ω*, as in Table 1. This indicates how much the axial force and then the seismic force can exceed the design value until the brace member reaches the complete plasticization.

Table 1. *NTC08 design criteria for steel CBF structures*

			X-Brace	Chevron Brace
Brace			Class sections 1 or 2; for circular hollow sections: $d/t \le 36$	
			$N_{Ed}/N_{pl,Rd} \le 1$ (1)	$N_{Ed}/N_{b,Rd} \le 1$ (2)
			$1,3 \le \bar{\lambda} \le 2$ (3)	$\bar{\lambda} \le 2$ (4)
			$\Omega = \Omega_{\min} = \left\{ \dfrac{N_{pl,Rdi}}{N_{Edi}} \right\}_{\min}$; $\dfrac{\Omega_{\max} - \Omega_{\min}}{\Omega_{\min}} \le 1,25$ (5)	
Beam			$N_{Ed} = N_{EdG} + 1{,}1\gamma_{Rd}\Omega_{\min}N_{EdE}$ (6)	 $\gamma_{pb} = 0{,}3$ (7)
Column			$N_{Ed} = N_{EdG} + 1{,}1\gamma_{Rd}\Omega_{\min}N_{EdE}$ (8)	
q-factor	Ductility Class	High	4	2,5
		Low	4	2

where: d and t are diameter and thickness of the circular hollow profile, respectively; N_{Ed}, $N_{pl,Rd}$, $N_{b,Rd}$ are the brace design axial force, plastic resistance, buckling resistance; $\bar{\lambda}$ is the brace normalized slenderness; Ω is the overstrength factor; γ_{Rd} is the steel overstrength factor that is the ratio between the average and the characteristic values of the yielding strength; γ_{pb} is a factor representative of the residual brace strength after buckling; N_{EdG}, N_{EdE} are the axial forces corresponding to non-seismic and seismic loads.

It is not the same for all diagonals, it depending on the distribution of internal forces within the structure and on some sources of oversizing, like the selection of structural members among the standard profiles or the need to provide lateral stiffness for deformability check, further to the imposed limitation of slenderness. The latter condition is particularly strict at the upper stories, giving rise to Ω factors increasing along the height of the structures. As a consequence, aiming at assuring a distribution in elevation as uniform as possible to promote the yielding of all braces, the difference between the maximum and the minimum values should be limited to 25% (Table 1). Moreover, considering that diagonals do not plasticize together at the same level of seismic forces, the Ω factor to be used is assumed as the minimum one, Ω_{min}, corresponding to the first not linear event, such as the plasticization of the first brace. Once designed the braces and calculated the Ω factor, the capacity design criterion is applied for determining the design forces for beams and columns (Eq. 8, Table 1).

2.2 CBF-X

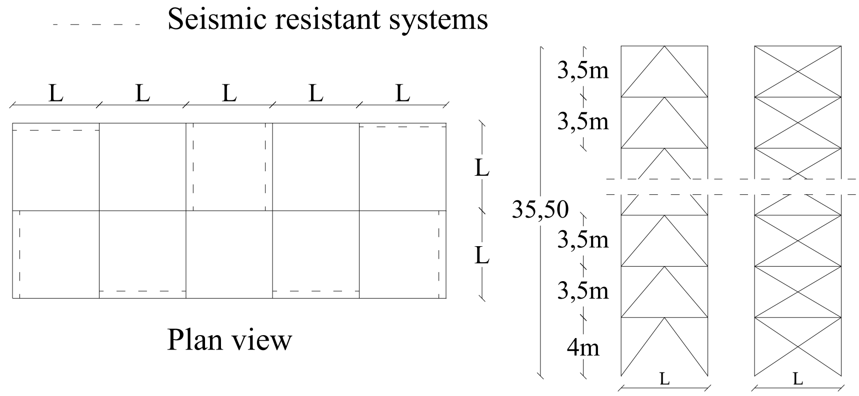
The design of CBF-X is performed by considering only the contribution of braces in tension, assuming that at collapse braces in compression are already buckled and do not provide any bearing capability. With this assumption, tensile braces are designed on the basis of the plastic resistance (Eq. 1, Table 1). Moreover, the normalized slenderness $(\bar{\lambda})$ of diagonals should be limited within a prefixed range (Eq. 3, Table 1), where the upper limit has the aim to avoid excessive distortions due to buckling of braces in compression, which could cause damage to connections or claddings, while the lower limit ensures the validity of the structural model with only active tensile braces as well as restricts the design internal forces in the columns, which are commensurated with the plastic resistance of braces. The behaviour factor q for dissipative structures is assumed as equal to 4 for both low and high ductility classes. In the ideal condition in which the whole brace in tension is plasticized, ductility and dissipation capability of members would be much greater than how quantified by such q value. However it is not possible to be confident on the ideal behaviour due to the uncertainties related to the behaviour of braces under the seismic cyclic actions. In fact braces undergo alternate states of tension and compression, therefore if the brace in compression buckles, the unstable deformed shape in bending is characterized by localized plastic deformation, thus the subsequent cycle in tension finds a degraded member, with limited ductile capabilities.

2.3 CBF-V

In the CBF-V, the compressed brace provides a contribution to the overall system stability, thus it cannot be neglected. For this reason, both braces are taken into account in the design model. Therefore the design resistance is the buckling strength (Eq. 2, Table 1) and only the upper limit of the normalized slenderness of diagonals is imposed (Eq. 4, Table 1). In addition, beams have to be designed by considering the concentrated force at the middle-span due to the unbalanced force between the plastic resistance of the tensile brace and the residual resistance of the compressed one after buckling, the latter being set equal to 30% of the brace plastic resistance (Eq. 7, Table 1). The beam is therefore subjected to bending moment, shear and axial forces. The behaviour factor q is equal to 2,5 and 2 for High and Low ductility classes, respectively.

3. THE CASE STUDY

The study structures have typical configuration and size of a regular building. The reference geometrical scheme, together with the main design features are given in Figure 1. For the sake of simplicity, the elastic spectrum is obtained according to the code OPCM 3431 [2005] since seismic parameters are independent from the geographic position, unlike the current NTC2008. Each case study is designed through either the Linear Static (LS) or Linear Dynamic (LD) analysis. For CBF-X the profiles used for the diagonal members are HE sections; in total 6 case studies are examined. For CBF-V for columns two cross section types are used, namely welded box sections and HE profiles, for beams HE profiles are used; while for braces two cross-section types, namely Circular Hollow Sections (CHS) and HE profiles, are used; in total 12 case studies are examined. As far as the Damage Limit State (DLS) is concerned, the limitation of the inter-story drift equal to 1% is considered, corresponding to infill panels not rigidly connected to the main structure.



Design features: 3, 6, 10 stories	*where:*
h = 3,5m (h_{GF} = 4m)	*h* is the interstory height,
L = 6m	h_{GF} is the interstory heigth at the ground floor,
High ductility class	*L* is the span,
a_g = 0,35g soil B	a_g is the peak ground acceleration,
G_k=4,81 kN/m^2	G_k is the characteristic value of the dead load,
$G_{k,T}$=5,20 kN/m^2	$G_{k,T}$ is the characteristic value of the dead load at the top story,
Q_k=2 kN/m^2	Q_k is the characteristic value of the live load.

Figure 1. *Geometry and design features of the study structures*

4. DESIGN ASSESSMENT

4.1 CBF-X

The NTC08 design procedure shows a first critical issue in the ambiguity in the use of LD analysis. For CBF-X, the code prescribes that at the ULS only 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 LD 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 the axial forces as the double of the one calculated by means of the structural model including both diagonals (Fig. 2).

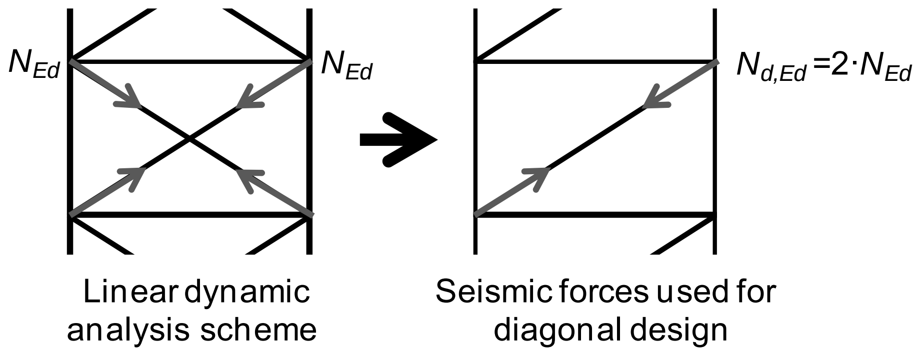


Figure 2. *Structural scheme assumed for linear dynamic analysis [Faggiano et al., 2014].*

In Table 2, the main design information on the study structures, such as W the structural weight, T the fundamental period of vibration, F_h the design base shear and Ω_{min} the design over strength factor, are reported. The design results show that the CBF-X designed by LS analyses are generally subjected to seismic actions higher than those designed by LD analyses. This difference is mainly related to the underestimation of the fundamental vibration period through the empirical formula provided by NTC2008 in case of LS analyses. This issue is more evident for taller buildings. For instance, in case of 10s structures, T calculated by the code formula is smaller than LD T, with a consequent increment of the total seismic force. This issue also influences the weight of the seismic resistant members, in fact, the structural weight of LS structures is higher than the LDs.

Table 2. *Design results for CBF-X [Faggiano et al., 2014]*

Design Method	N. storeys (s)	W [kN]	T [s]	F_h [kN]	Ω_{min}
LS	3	37	0.30*	348	2.52
	6	110	0.50*	687	1.88
	10	186	0.73*	781	1.44
LD	3	37	0.31	354	2.40
	6	95	0.59	548	1.94
	10	149	1.25	486	1.75
*$T=C_1H^{3/4}$ with C_1 = 0.05, H= total height of the structure					

Another critical issue observed in the design phase is the difficulty in selecting the bracing profiles. In particular, the lower bound of $\overline{\lambda}$ (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 Ω values. This especially occurs at the top storey, where the uniformity of the Ω factor distribution along the structure height is hard to be satisfied (Eq. 3, Table 1). Thus, for the 10s structures examined the top storey has not been considered in the Ω check.

4.2 CBF-V

Commonly, the design of columns is conditioned by the gradual cross-section reduction criterion. In fact, strong variation of the cross-section sizes along the building height should be avoided, they being generally a source of localised damage. Moreover the structure story stiffness variation at consecutive floors should be limited, in order to fulfil regularity requirements devoted to assure the achievement of the most uniform state of stress and deformation and in particular plastic hinges distribution along the structure height at the ultimate limit states. Another influencing design aspect is that HE profiles larger than HEB300 are barely able to withstand high axial loads, because, as far as the depth increases, the base is almost constant; thereby, being the increment of second moment of area extremely limited, for profiles larger than HEB300, the axial buckling check is hard to be satisfied. For these reasons, for 10s structures and 6s HE braces structures, the columns are realized with welded square box sections, opportunely reduced along the building height, in order to absorb the high axial loads deriving from the capacity design criterion. Columns are HE profiles for 6s frames in case of CHS braces, as the same for 3s frames, where, due to the limited number of floors, the hierarchy design criterion is not penalising. In Table 3 the main design information on the study structures are reported. In Figure 3 the response spectra with the evidence of *T* for 3s, 6s and 10s in case of HE braces and CHS braces are shown. In general some observations come be done by comparing on one hand CHS and HE braces structures, on the other hand LS and LD design. CHS braces structures are lighter than HE braces structures. This is particularly evident for LS 10s buildings, where the adoption of HE braces induces the use of welded double T beam profiles at the lower storeys, due to the high forces transferred by the braces, oversized for the limited availability of standard HE hot-rolled profiles. This limitation does not exist for CHS profiles, which are produced with a large range of cross-sections.

Table 3. *Design results for CBF-V [Faggiano et al., 2015b]*

Design Method	N. storeys	W [kN]		T [s]		F_h [kN]		Ω_{min}	
		CHS	HE	CHS	HE	CHS	HE	CHS	HE
LS	3	46	63	0.3*	0.3*	549	549	2.15	3.90
	6	*159*	*234*	*0.5**	*0.5**	*1085*	*1085*	*2.11*	*4.04*
	10	408	722	0.73*	0.73*	1254	1254	2.46	4.43
LD	3	52	71	0.33	0.25	580	588	2.32	4.21
	6	*140*	*215*	*0.58*	*0.44*	*1062*	*1064*	*1.86*	*3.41*
	10	309	602	1	0.68	988	1428	2.34	3.70
$*T=C_1H^{3/4}$ with C_1 = 0.05, H= total height of the structure									

The vibration periods *T* determined by LD analysis are higher for CHS braces structures than for HE braces structures, in conformity to the previous observation. In the LS design the first vibration period does not depend on the bracing details, it being calculated by means of the simplified formula. Moreover, the LD *T*, as respect to the LS *T* is lower for HE braces structures, is higher for CHS braces structures. In case of 10s and 6s structures the LD design gives rise to lower base shears and weights as respect to the LS design, contrary in case of 3s frames.

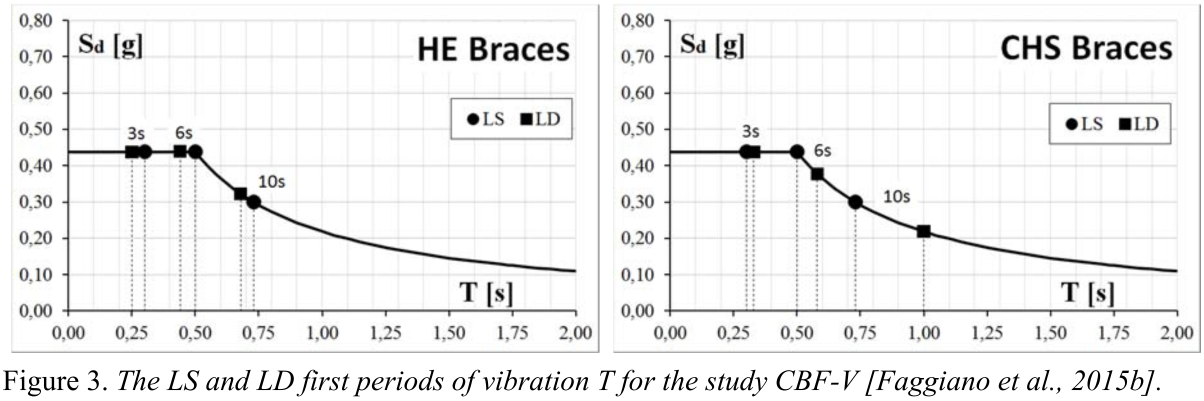


Figure 3. *The LS and LD first periods of vibration T for the study CBF-V [Faggiano et al., 2015b].*

This trend does not reflect the variation of the first periods of vibration *T*, due to the influence of the superior modes of vibration, which could provide not negligible additional actions that are ignored by the LS analysis.

Concerning the over-strength factor the following observations can be done:

- High values of Ω_{min}, ranging from 1,86 to 2,46 and 3,41 to 4,43 for CHS and HE braces structures, respectively, are achieved. These imply a significant increment of design axial forces in the columns according to the capacity design.
- The Ω variation ratio (Eq. 5, Table 1) is always governed by the top storey braces, whose Ω values are generally larger than those at lower storeys. This is due to the use of brace cross-sections, which are subjected to low seismic actions but should contemporary respect the standard slenderness limit. As a consequence, elastic members are oversized, it producing a weight increase.
- In case of HE braces, Ω_{min} is generally larger than the design behaviour factor q (2,5), what is not acceptable.

5. CONCLUSIONS

Based on the previous observations possible improvements of the NTC2008-EC8 design criteria for CBF can be related to the following items:

1. A possible improvement of the design criteria for CBF-X could be to clearly state the design procedure in agreement with the ULS model with only tensile diagonal active.
2. The use of HE profiles for structural members can become more convenient if a wider spectra of cross sections are produced, in order that profiles could best fit all the design requirements in terms of strength and stiffness, reducing the over strength that alters the effect of the design provisions as respect to the expectations. This could also either avoid the scatter in the geometrical variability of members composed by different parts, as it occurs for columns belonging to high rise buildings, or enhance the efficiency of the capacity design, or simplify the connection among members.
3. The simplified formula for the determination of the first period of vibration, necessary in case of LS design, should be better fitted according to the number of floors, taking also into account the structural system type, it being differentiated in case of bracing systems and for type of bracings.
4. The top story needs specific design criteria, which balance capacity design and slenderness requirements. Some authors proposed a different approach based on the reduction of the bracing members section at the ends to obtain Ω=1 [Giugliano *et al.*, 2010; 2011].
5. With regards to capacity design, it should be explicitly stated that the over strength factor Ω should be in any case lower than the design behaviour factor q.

In general, results briefly presented in the current paper, in line with the literature references, delineate some important issues and suggestions for improvements, related to the design procedure and structural models. However they require more wide elaborations through further extensive campaign of both experimental and numerical investigations aiming at both optimizing the calculation models and providing simplification to the design methods.

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7. REFERENCES

D'Aniello M., La Manna Ambrosino G., Portioli F., Landolfo R., [2013]. Modelling aspects of the seismic response of steel concentric braced frames. *Steel and Composite Structures, An International Journal*, Vol. 15, No. 5, 539-566 November 2013.

D'Aniello M., Costanzo S., Landolfo R., [2015]. The influence of beam stiffness on seismic response of chevron concentric bracings. *Journal of Constructional Steel Research*. 112: 305-324. Doi: 10.1016/j.jcsr.2015.05.021.

Decree of the Minister Council Presidency 03/05/2005 n. 3431 (OPCM) [2005] Further modifications and integrations to the Decree of the Minister Council Presidency 20/03/2003 n. 3274 (in Italian).

EN 1993-1-1 [2005] Eurocode 3: Design of steel structures Part 1-1: General rules and rules for buildings, European Committee for Standardization, Bruxelles.

EN 1998-1 [2005] Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings, European Committee for Standardization, Bruxelles.

Faggiano, B., Fiorino, L., Formisano, A., Macillo, V., Castaldo, C., Mazzolani, F.M. [2014] "Assessment of the design provisions for steel concentric X bracing frames with reference to Italian and European codes", *The Open Construction and Building Technology Journal* **8**, (Suppl 1: M3), 208-215.

Faggiano B., Fiorino L., Formisano A., Castaldo C., Macillo V., Mazzolani F. M. [2015a]. Appraisal of the design criteria for concentric braced steel structures according to Italian and European codes. In XXV Congresso C.T.A. Le giornate Italiane della costruzione in acciaio, 1-3 Ottobre 2015, Salerno, ISBN 9788894008944, pp.445-452.

Faggiano, B., Formisano, A., Fiorino, L., Castaldo, C., Macillo, V., Mazzolani, F.M. [2015b] "Assessment of the design criteria for concentric V-braced steel structures according to Italian and European codes". *The Open Civil Engineering Journal*, Betham Open. (in press).

Formisano, A., Faggiano, B., Landolfo, R., Mazzolani, F.M. [2006] "Ductile behavioural classes of steel members for seismic design", In: Mazzolani, F.M., Wada. A. *Behaviour of Steel Structures In Seismic Areas*, p. 225-232, London: Taylor & Francis Group/ Balkema, ISBN: 9780415408240, Yokohama, 14-17 August.

Giugliano, M. T., Longo, A., Montuori, R., Piluso, V. [2010] "Plastic design of CB-frames with reduced section solution for bracing members", *Journal of Constructional Steel Research* 66 (5), 611-621.

Giugliano, M. T., Longo, A., Montuori, R., Piluso, V. [2011] "Seismic reliability of traditional and innovative concentrically braced frames", *Earthquake Engineering and Structural Dynamics*, 40 (13), 1455-1474.

Longo, A., Montuori, R., Piluso, V. [2008a] "Failure mode control of X-braced frames under seismic actions", *Journal of Earthquake Engineering* 12 (5), 728-759.

Longo, A., Montuori, R., Piluso, V. [2008b] "Plastic design of seismic resistant V-braced frames", *Journal of Earthquake Engineering* 12 (8), 1246-1266.

Marino, E. M. [2014] "A unified approach for the design of high ductility steel frames with concentric braces in the framework of Eurocode 8", *Earthquake Engineering and Structural Dynamics* 43, 97–118, DOI: 10.1002/eqe.2334, 2014.

Mazzolani, F.M., Della Corte, G. [2008] "Steel structures in seismic zone – M.D. 2008" (in Italian), *Il Sole 24 Ore* (ed.).

Mazzolani, F., Landolfo, R., Della Corte, G., Faggiano, B. [2006] *Steel structures buildings in seismic zones* (in Italian), Pavia: Iuss Press, ISBN: 9788873580348.

Ministerial Circular 02/02/2009 n. 617 (M.C.) [2009] Instructions for application of the "New technical codes for constructions" (in Italian).

Ministerial Decree 14/01/2008 (M.D.) [2008] *New technical codes for constructions* (in Italian).

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VALUTAZIONE DEI CRITERI DI PROGETTO PER LE STRUTTURE DI ACCIAIO A CONTROVENTI CONCENTRICI SECONDO LE NORME ITALIANE ED EUROPEE

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SOMMARIO: *La revisione critica dei metodi di progetto presenti nelle norme tecniche italiane per le costru-zioni NTC2008, allineate con l'Eurocodice 8, per le strutture a controventi concentrici con diagonali a X e a V inversa, ha lo scopo di definire criteri di progetto più efficaci nel garantire gli adeguati livelli di sicurezza sismica. I casi di studio sono rappresentativi di tipiche configu-razioni strutturali di CBF. Essi sono progettati secondo le NTC2008, mediante analisi lineari sia statiche sia dinamiche. L'obiettivo è l'identificazione delle carenze delle regole di progetto, con particolare riferimento sia all'applicabilità delle procedure proposte sia alla reale possibili-tà di dimensionare le diagonali e le membrature strutturali ad esse collegate, quali travi e co-lonne. La discussione sui risultati ottenuti ha consentito di evidenziare gli aspetti sia positivi sia negativi dell'approccio progettuale attuale e di proporre alcuni miglioramenti dei criteri di progetto.*

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