Available online at www.sciencedirect.com

ScienceDirect

Procedia Engineering 199 (2017) 3522–3527

Procedia Engineering

www.elsevier.com/locate/procedia

X International Conference on Structural Dynamics, EURODYN 2017

Linear Time-History Analysis for EC8 design of CBF structures

Alessia Di Cuia[a,b], Luca Lombardi[a], Flavia De Luca[a,]*, Raffaele De Risi[a], Silvia Caprili[b], Walter Salvatore[b]

[a]*Department of Civil Engineering University of Bristol, University Walk, Bristol BS8 1TR, United Kingdom*
[b]*Department of Civil and Industrial Engineering University of Pisa, Largo Lucio Lazzarino, Pisa 56122, Italy*

Abstract

Increased availability of strong motions, improved computational capabilities and availability of open-source software for Eurocode 8-compliant record selection make Linear Time History Analysis (LTHA) a potential more rigorous analysis approach for the design of structures. This methodology is not explicitly codified in the Eurocode and it requires some "code-friendly" recommendations for its implementation in practice. Herein, LTHA is employed for the first time as design method for a five-storey archetype high ductility class concentric brace frame (CBF). A new-optimized design method for braces is presented for the smooth implementation of both Response Spectrum Analysis (RSA) and LTHA. In fact, the tension-only design approach for CBF in Eurocode 8 has some criticalities arising even when a routine RSA is employed. The design process is rather smooth only in the case of linear static approach classically implemented for regular, mid-rise structures.
Results of this study provide insights for validation of LTHA as design method for CBFs, and for its code implementation as routine design method in EC8, valuable in near source conditions or for irregular structural configurations.

Peer-review under responsibility of the organizing committee of EURODYN 2017.

Keywords: Eurocode 8, Linear Time-History Analysis, Concentric Brace Frames, Tension-Only design.

1. Introduction

Concentric Brace Frame (CBF) are a class of structures resisting lateral loads through concentric system subjected to axial forces. During an earthquake, CBFs are expected to yield and dissipate energy through post-buckling hysteretic behaviour of bracing members [1].

* Corresponding author. Tel.: +44(0)1173315296
E-mail address: flavia.deluca@bristol.ac.uk

1877-7058 © 2017 The Authors. Published by Elsevier Ltd.
Peer-review under responsibility of the organizing committee of EURODYN 2017.
10.1016/j.proeng.2017.09.502

The design of CBFs is dealt in two different ways (i.e., tension-only, and compression-only) [2]. In this study, a tension-only philosophy is adopted, in accordance with Eurocode 8 (EC8) design prescriptions [3]. In the tension-only design, the storey shear is entirely resisted by the tension braces. Conversely, the United States (US) code [4] is based on the compression-only idealisation. The compression-only design approach accounts for the buckling capacity of the braces in compression [2]. The design of CBFs is typically carried out through equivalent static analysis and, then, verified through nonlinear analysis approaches [5]; on the other hand, in general, the routine design approach for EC8-conforming structures would be a linear dynamic analysis: the so-called response spectrum analysis.

In the linear static analysis or equivalent static analysis, the effect of the earthquake on the building is represented by a system of horizontal forces, applied statically to the structure and distributed at the various floors, with an inverted triangular shape (first-mode). This schematisation can be adopted if the construction is regular in height and the period of the main vibration mode (T_1), in the direction in examination, does not exceed $2.5T_C$ or T_D (where T_C and T_D are the boundary of the constant velocity branch of the elastic spectral shape according to EC8). This method is not suitable for tall buildings, where second and higher modes can be important, or for structures with any irregularity in plan or elevation. When the conditions on the period and regularity are not met the Response-Spectrum Analysis (RSA) is the reference method for EC8. Strictly speaking, RSA is an approximate approach for the evaluation of linear dynamic response of structures. It accounts for the effects of higher modes through the approximate combination of results of static analyses defined on the basis of the modal properties of the structure. The Complete Quadratic Combination (CQC) is the combination rule for RSA since 1980s [6], and it is suggested as the main combination approach by many codes, e.g., EC8 [3] and the Italian Seismic code [7]. It presents some limits for building in near source condition subjected to impulsive earthquakes and for very stiff structures [8]. Moreover, the combination rule of RSA results in a loss of sign of the earthquake actions, and this aspect can become critical for the design of non-symmetric or irregular CBF frames where knowing the value of compression and tension configuration can be important. The RSA is applied to the 3D model of the structure in which both diagonals in tension and compression should be included. If the structure is simple and regular, the identification of the compressed brace is straightforward and the subsequent tension-only design can be performed with a linear static approach. The design of CBF structures through RSA needs some additional recommendations and different studies in the past tried to address it (e.g., [9]). The issue is that often such procedures ends up in "non-design-friendly-approaches".

Linear Time History Analysis (LTHA) can be an appealing alternative to RSA now that accelerograms are more available, computational capabilities are improved, and open-source software for EC8-compliant record selection are available (e.g., [10]). Applied to a linear model of the structure, LTHA is a step-by-step procedure where the loading and the response histories are evaluated at each time step. It considers the interaction of the modes of vibration with the typical frequencies of an earthquake defined by accelerometric waveforms rigorously. Notwithstanding, its use in some specific professional applications, such as design of dams and other critical structures, LTHA does not have a codified procedure in EC8 and in other codes. Some efforts were recently done on this aspect towards the formulation of an EC8 compatible LTHA (e.g., [11]). The employment of LTHA for design of CBF solves some of the issues related to the exact configuration of tension and compression in the structure, but it does not solve the problem of the tension-only design approach that takes for granted a "modified" configuration of the structure. EC8 design of CBF is critical for both RSA and LTHA applications. In this study, a first attempt towards an optimized and easy-to-implement design method for CBF is presented. Such optimized design is applied to a five-storey steel CBF archetype building in High Ductility Class and compared with the design results obtained with the traditional static design and the RSA-based procedure proposed by Martinelli et al. [9]. This first application to a regular structure allows the definition of a refined methodology for more complex design situations.

2. EC8 design of CBFs through Linear Analyses: a new approach

The quality of the seismic response of CBFs is determined by the performance of the brace. The requirements regarding the steel cross sections which dissipate energy depend on the ductility class and the behaviour factor q. EC8 design of CBFs in High Ductility Class (DCH) requires Class 1 cross-sections allowing the assumptions of q equal to 4 or higher. In the case of Class 2 sections, the design of CBFs has to be downgraded to Medium Ductility Class (DCM) with the assumption of q between 2 and 4 [3,12]. Cross-section classes depend on the width to thickness ratio of the parts subject to compression and/or bending [12].

The CBF seismic design follows the capacity design philosophy. EC8 imposes an upper limit (equal to 2.0) for the non-dimensional slenderness of braces ($\overline{\lambda}$) in order to prevent the rapid degradation of braces' resistance. Such a limit is additional with respect to the prescription of EC3 for the lower limit of $\overline{\lambda}$ (equal to 1.3) used in static design situations to avoid overloading columns in pre-buckling stage. The tension-only designed braces according to EC8 have to respect upper and lower $\overline{\lambda}$ limits. Moreover, in order to ensure a homogeneous dissipative behaviour of the brace system, EC8 requires that the overstrength factor (Ω) has to be uniform along the different storeys. In fact, according to EC8, the maximum relative increase ($\Delta\Omega$) has to be less than 25% and evaluated as the relative error between the maximum ($\Omega_{\max\{i\}}$) and the miminum ($\Omega_{\min\{i\}}$) overstrength among all storeys, see Eq. (1), where i indicates the i^{th} storey, N_{Ed} is the design value of the axial force, and $N_{pl,Rd}$ is the yield resistance of the steel member.

$$\Omega = \frac{N_{pl,Rd}}{N_{Ed}}; \quad \Delta\Omega = \frac{\Omega_{\max\{i\}} - \Omega_{\min\{i\}}}{\Omega_{\min\{i\}}} \leq 0.25 \quad i = 1,..n \tag{1}$$

The limitations on $\overline{\lambda}$ and $\Delta\Omega$ are very restrictive. In compliance with these conditions, seismic design may lead to oversized and very heavy structures. The satisfaction of the upper slenderness limit inevitably results in the selection of larger cross sections; consequently increasing Ω, especially in the upper storeys, where the seismic demand is low [13]. Furthermore, the minimum value of Ω multiplies the axial force acting on beams and columns, in accordance with capacity design provision in EC8 [3]. High Ωs cause over-sizing of the non-dissipative structural members. On the last two top storeys, it is possible to neglect the upper limit of $\overline{\lambda}$ [14]. The evaluation of $\Delta\Omega$ can be done excluding the value of Ω resulting from the top storey [15], and making feasible the selection of commercial brace profiles.

As long as the structure is regular in plan and elevation [3], the design of X bracing can be easily performed through linear static analysis. As shown in Fig. 1, the structural analysis is done considering only the tension diagonals in each braced bay. When the structure does not meet the regularity requirements, the design should be carried out through RSA. However, in the case of CBFs, this analysis is critical because of the loss of signs in braces and the consequent difficulty in identifying the compressed member to be removed in the tension-only configuration.

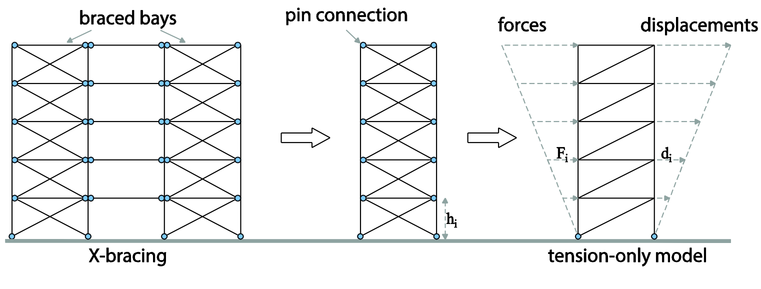

Fig. 1. Schematic representation of tension-only analysis of CBF (adapted from [16])

Martinelli et al. [9] defined a design approach with RSA, which applies the "tension-only" concept by employing sets of static equivalent seismic forces computed from RSA. They suggest performing a RSA on the 3D model with both braces with reduced section and then defining tension-only plane models to be subjected to different modal distributions of forces for the final CQC combination of the results aimed at the tensile design of the braces [17]. The design approach suggested by Martinelli et al. partially solves issues of RSA for CBFs. On the other hand, the algorithm is complex and it would be very time-consuming in case of tall or irregular structures. In lieu of Martinelli et al.'s method, another optimized design approach for steel CBFs is proposed herein. The new methodological approach takes into account the contribution not only of the brace in tension, but also of the compressed one, as long as it does not attain its buckling strength ($N_{b,Rd}$), the dotted line in Fig. 2b. The RSA is carried out on the full 3D model. The general principle is that as soon as the compressed element attains its buckling resistance, it is not able to carry any additional axial force. The exceeding compression force (N_j - $N_{b,Rd}$) is assigned as additional tension load to that obtained from the analysis (N_i') and this final tension value (N_i) is employed for the dimensioning of the brace, see Eq. (2) and Fig. 2. This optimized design methodology is very general and it can be applied either in the case of RSA or LTHA. For LTHA, it provides an even more efficient design approach; in fact, in the case of LTHA, it explicitly accounts for tension and compression experienced step-by-step by the brace members.

$$N_i = N_i^{'} + (N_j - N_{b,Rd}) \tag{2}$$

The LTHA does not have yet a strictly codified procedure in EC8. Recently, efforts have been made to carry out a robust code-oriented procedure for it, and a preliminary approach was validated for the case of Reinforced Concrete structures [12]. The principle is to make the input selection in analogy to what is done for nonlinear analyses, using, for example, a set of minimum seven spectrum-compatible ground motion couples [10], and to apply q as downscaling factor to each accelerogram. The application of LTHA needs a bespoke definition of q, and it still does not have any robust simplified approach to take into account the accidental eccentricity, as it is done in static design methods [12]. It is worth noting that the issue of accidental eccentricity is still open also for the nonlinear dynamic analyses. The application of the optimized method to LTHA results in a design tension envelope (green line in Fig. 2) for each brace and each single couple of ground motion combinations considered in the analysis.

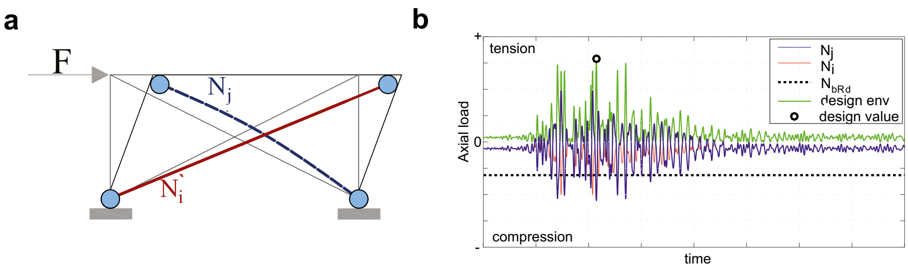

Fig. 2. Optimized design method applied to LTHA (a) Couple of braces' behaviour; (b) Tension envelope result.

3. Case study

The archetype five-storey CBF structure has five 6m bays in X direction and three 7m bays in Y direction. Interstorey height is 350 cm and the braced bays are those in the corners of the plan (see Fig. 3). The building is for office occupancy and the floor permanent and live loads were calculated considering predalles precast floor slab (G_1=3.70kN/m^2). Snow load is 1.40kN/m^2, while for wind loads a terrain category II was considered [18].

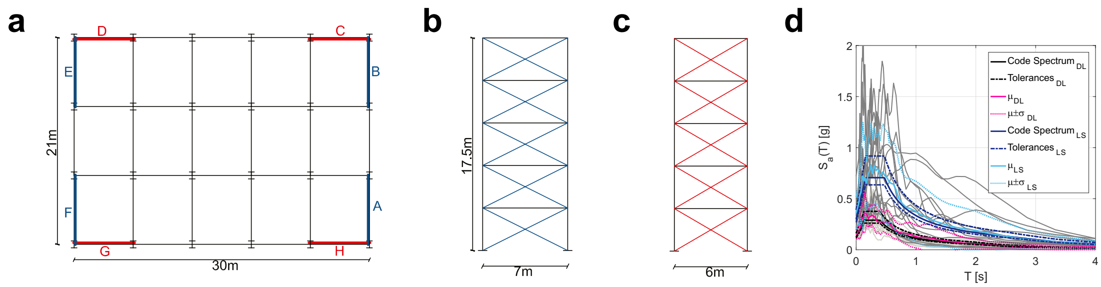

Fig. 3. Archetype 5-storey CBF structure (a) Plan with braced bays; (b) Elevation scheme x direction, (c). Elevation scheme y direction, (d) code spectra for Damage Limitation (DL) and Life Safety (LS) limit states and the two sets of sever pairs of accelerograms selected for each limit state

The building is located in L'Aquila (Italy), on class B soil. The elastic design spectra with return periods of 475 and 50 years are shown in Fig. 3d, respectively. The anchorage ground acceleration (a_g) for life safety limit state (475 years) is equal to 0.27g. LTHA is based on a set of seven accelerograms per limit state considered in the design (see Fig 3d); it is the minimum required by EC8 to compute the mean response of the 7 couples of time-histories. Records are selected through the freeware software Rexel [10], and they are applied as 14 different combinations on the structure inverting the two components of the records and the directions of application along the building. Such approach would request a higher number of combination that is reduced to 14 because of the symmetry of the building and the fact that at this stage in all the designs done accidental eccentricity was discarded [12].

Fig 4 shows the comparison of the dynamic methods discussed in section 2 with respect to the static analysis in terms of relative error of design axial load (e_N) for braces at each storey. The reduction in terms of design axial load is very significant when a dynamic method is employed; LTHA, in particular, leads to a reduction from 40% to 70% of the mean axial load. The significant reduction in design axial load for braces has a counterpart in the increase of Ω for the design of columns as shown in Fig 5. Static analysis allows a better control of Ω in the design keeping its value very close to 1.

Table 1 shows the design results obtained through the four different approaches considered. The braces' configuration resulting from static analysis seems to be overdesigned if compared with any of the three alternative design methods. Martinelli et al., and the RSA and LTHA design approach introduced in this study result all in the same brace configuration for both directions. The difference between the three designs arises in the dimensioning of columns at first storey as a result of the different Ωs obtained in each analysis (see Fig. 5). Martinelli et al. allows keeping the same columns' configuration of the static analysis, while the optimized RSA needs an increase of the columns' sections in the corners. Finally, the optimized LTHA requests for HEM300 in all the columns of the braced bays in x direction.

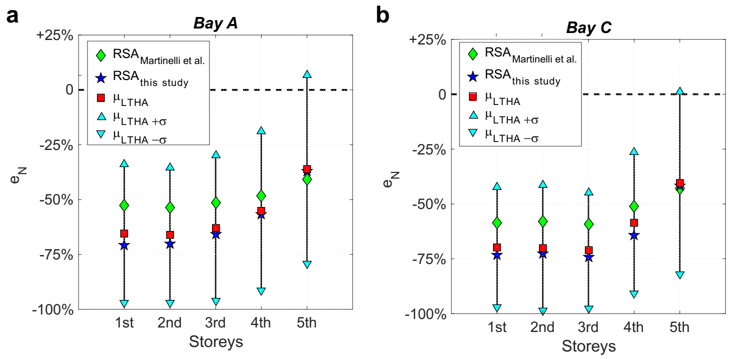

Fig. 4. Relative difference of design axial load (e_N) for braces at each storey between RSAs and LTHA with respect to static analysis for (a) Bay A in x direction and (b) Bay C in y direction. For the case of LTHA the mean and the mean ±one standard deviation is shown.

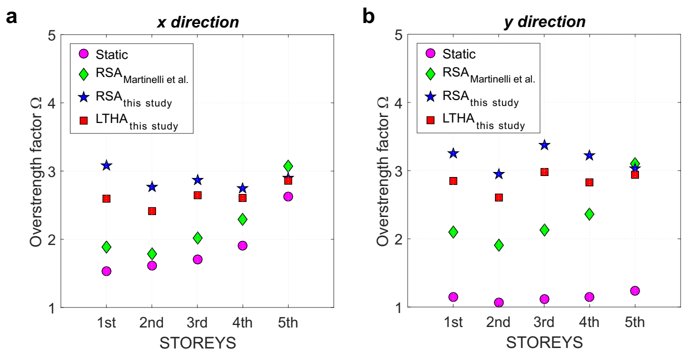

Fig. 5. Comparison of Ω in (a) x direction and (b) y direction used for design in the static and dynamic methods (Static, RSAs, and LTHA).

Table 1. Archetype 5-storey CBF structure designed according to different methodologies

Storey (i^{th})	Beams[1]				Columns[1]		Braces[2]	
	X		Y					
	external	internal	braced	non-braced	external	internal	static	other designs
1	HEA300	HEA400	HEA340	HEA220	HEM280[3]/HEM300[4,5]	HEM240	UPE270	UPE220
2	HEA300	HEA400	HEA320	HEA220	HEM240	HEM220	UPE240	UPE200
3	HEA300	HEA400	HEA300	HEA220	HEM200	HEM200	UPE220	UPE200
4	HEA300	HEA400	HEA260	HEA220	HEM200	HEM200	UPE180	UPE180
5	HEA260	HEA320	HEA220	HEA220	HEM200	HEM200	UPE100	UPE140

[1]S235; [2]S355; [3]all external for static and Martinelli et al. designs; [4]increased from HEM280 for corner columns in RSA optimized design; [5]increased from HEM280 for braced-bay columns along x in LTHA design.

4. Conclusion

A new optimized design approach is proposed and implemented with linear time-history analysis as methodology for design of EC8-conforming CBF structures allowing a more insightful application of the tension-only approach implemented by the code and being appealing for all the cases in which linear static and response spectrum analyses can lead to inaccurate outcomes. The new method is applied preliminarily for a regular 5-storey archetype structure. The new design method allows a significant reduction for brace sections at the cost of an increasing overstrength factor leading, in some cases, to an increase of columns' sections.

References

[1] Bruneau, M., Uang, C. M., & Sabelli, S. R, Ductile design of steel structures, McGraw Hill Professional, 2011.

[2] Elghazouli, A. Y., Seismic design procedures for concentrically braced frames, Proceedings of the Institution of Civil Engineers-Structures and Buildings, 156(4), 381-394, 2003.

[3] Comité Européen de Normalisation, Eurocode 8: design of structures for earthquake resistance—Part 1: general rules, seismic actions and rules for buildings. European Standard EN 1998-1:2003, Brussels, 2003.

[4] American Institute of Steel Construction (AISC), Specifications for structural steel buildings, Chicago, 2010.

[5] Braconi, A., Caprili, S., Degee, H., Guendel, M., Hjiaj, M., Hoffmeister, B., ... & Somja, H. Efficiency of Eurocode 8 design rules for steel and steel-concrete composite structures. Journal of Constructional Steel Research, 112, 108-129, 2015.

[6] Wilson, E. L., Der Kiureghian, A., & Bayo, E. P., A replacement for the SRSS method in seismic analysis, Earthquake Engineering & Structural Dynamics, 9(2), 187-192, 1981.

[7] DM 14.01.08, Nuove Norme Tecniche per la Costruzione, 2008.

[8] De Luca F, Verderame GM, The accuracy of CQC and response spectrum analysis in the case of impulsive earthquakes, Proceedings of the 11th International Conference on Structural Safety and Reliability ICOSSAR 2013, New York, USA, 2013.

[9] Martinelli, L., Mulas, M. G., & Perotti, F., The seismic response of concentrically braced moment-resisting steel frames, Earthquake engineering & structural dynamics, 25(11), 1275-1299, 1996.

[10] Iervolino, I., Galasso, C., & Cosenza, E., REXEL: computer aided record selection for code-based seismic structural analysis, Bulletin of Earthquake Engineering, 8(2), 339-362, 2010.

[11] De Luca F, Lombardi L, EC8 design through linear time history analysis versus response spectrum analysis – Is it an enhancement for PBEE?, 16th World Conference on Earthquake, 16WCEE 2017, Santiago Chile, 2017.

[12] Eurocode, C.E.N., Design of Steel Structures, Part 1-1;"General Rules and Rules for Buildings". CEN, Brussels: European Committee for Standardization, 2005.

[13] Brandonisio, G., Toreno, M., Grande, E., Mele, E., & De Luca, A., Seismic design of concentric braced frames. Journal of Constructional Steel Research, 78, 22-37, 2012.

[14] Bisch, P., Carvalho, E., Degee, H., Fajfar, P., Fardis, M., Franchin, P. & Somja, H., Eurocode 8: seismic design of buildings worked examples. Luxembourg: Publications Office of the European Union, 2012.

[15] Faggiano, B., Fiorino, L., Formisano, A., Macillo, V., Castaldo, C., & Mazzolani, F. M., Assessment of the design provisions for steel concentric x bracing frames with reference to italian and european codes, Open Construction and Building Technology Journal, 8(1), 208-215, 2014.

[16] Günaydın, E., & Topkaya, C. Fundamental periods of steel concentrically braced frames designed to Eurocode 8. Earthquake Engineering & Structural Dynamics, 42(10), 1415-1433, 2013.

[17] Martinelli, L., Mulas, M. G., & Perotti, F., The seismic response of concentrically braced moment-resisting steel frames, Earthquake ngineering & structural dynamics, 25(11), 1275-1299, 1996.

[18] En, B. S.. 1-4: 2005 Eurocode 1: Actions on structures—General actions—Wind actions, 1991.