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# Seismic behavior of concentrically braced frames designed to AISC341 and EC8 provisions

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ABSTRACT

Steel concentrically braced frames (CBFs) are frequently used as efficient lateral load resisting systems to resist earthquake and wind loads. This paper focuses on high seismic applications where the brace members in CBFs dissipate energy through repeated cycles of buckling and yielding. Widely-used seismic provisions have somewhat different approaches for the seismic design of CBFs. The present study evaluates in detail the similarities and differences between the design philosophies and provisions used in the United States and Europe for these systems. The requirements of both provisions applied during a full design procedure are summarized and compared. Furthermore, X-braced, split X-Braced, and V-braced archetypes are designed accordingly and the differences in the design outcomes are investigated regarding section sizes and the weight of steel used in each design. Finally, inelastic structural models of the designed archetypes are developed and subjected to a large set of ground motions to study their seismic behaviors. The results of a total of 880 nonlinear time history analyses are then synthesized to investigate the way in which the requirements of these provisions affect the seismic behavior of the designed CBF. Notable differences are observed between the performances of the CBFs designed using American and European provisions. The similarities and differences as well as drawbacks of the provisions are thoroughly discussed. Recommendations and future research needs are suggested to enhance the seismic performance of steel CBFs designed according to these provisions.



## 1. Introduction

Currently, moment resisting frames, concentrically braced frames, eccentrically braced frames, special truss moment frames, steel plate shear walls, and buckling restrained braced frames are being commonly used as lateral (i.e. seismic and wind) force resisting systems for steel structures. While new systems such as buckling restrained braced frames are gaining popularity, moment resisting frames (MRFs) and concentrically braced frames (CBFs) are considered as two of the most popular systems among these alternatives. Although MRFs provide more architectural freedom, compared to the CBFs, they are expensive. CBFs have been quite popular since the 1960s mainly because of their economic advantages over MRFs particularly in cases where the drift requirements govern the design. Furthermore, beam-to-column connections of MRFs suffered premature fractures in the 1995 Kobe and the 1994 Northridge earthquakes [1,2]. In the aftermath of these earthquakes, considerable research and development projects were conducted in the US, Japan, Europe, and elsewhere to develop new moment connections that have sufficient strength, stiffness, and ductility to perform satisfactorily during future strong seismic events. However, the new MRF connections and the modifications made to then existing moment connections have caused their cost of construction and inspection to increase significantly, making the use of CBFs even more economical. More recently, the 2011 Christchurch earthquake in New Zealand resulted in fracture of several eccentrically braced frames (EBFs), further adding to the popularity of CBFs. The CBF system is currently one of the most widely used seismic load resisting systems in steel structures; it is easy to design and the most efficient especially in controlling lateral drifts of buildings.

In recent decades, a significant amount of research has been conducted on the seismic behavior and design of CBFs. A major portion of these studies has focused mainly on the response of bracing members and their connections [3–18]. Extensive experimental [19–30] and numerical [31–38] investigations have also been undertaken to study the behavior of single-story and multi-story CBFs under severe loading scenarios, assessing both the system level and component level responses.

In the United States, steel CBFs are designed according to the AISC Specification for Structural Steel Buildings [39], hereafter referred to as AISC360, as well as the special seismic design rules of the AISC Seismic Provisions for Structural Steel Buildings [40], which is referred to here as AISC341. In Europe, CBFs are designed according to the regulations

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of the Eurocode 3: Design of Steel Structures – Part 1-1: General Rules and Rules for Buildings [41], hereafter referred to as EC3, as well as the seismic provisions of the Eurocode 8: Design of Structures for Earthquake Resistance – Part 1: General Rules, Seismic Actions and Rules for Buildings [42], which is referred to here as EC8.

Due to rapid globalization, engineers are now faced with the challenge of being competent with several design provisions. Owners may require the use of widely accepted design codes regardless of the location of the structure. Nevertheless, in some cases, there can be substantial dissimilarities between the regulations of different provisions which might significantly affect the behavior of the designed structure.

## 2. Objectives and scope

Seismic design provisions in AISC341 and EC8 on MRFs and EBFs are quite similar. However, the rules on the seismic design of CBFs in these provisions have evolved separately and have some significant philosophical as well as procedural differences. The main objective of this paper is to study the similarities and differences between the practices in the United States and Europe regarding the seismic design of CBFs. Under this goal, the provisions given in AISC341 and EC8 for steel CBFs are compared and studied thoroughly in Section 3. To illustrate the way in which the requirements of these provisions influence the final structure and its seismic behavior, CBF archetypes are designed in Section 4 following the regulations of these two provisions. X-braced, split X-braced, and V-braced configurations are considered. Section 5 has the data and results of the designed archetype structural models subjected to a large set of ground motions as well as the observations made on the responses. Clear comparisons between the seismic performances of the CBFs designed according to the provisions are made with the aid of graphical presentation of the results. Similarities and differences between the behaviors are highlighted and recommendations are developed for practicing engineers.

## 3. Comparison of design provisions in AISC341 and EC8

### 3.1. Definition and geometries

AISC341 and EC8 provisions both define CBFs as systems where horizontal forces are mainly resisted by members subjected to axial forces. The centerlines of adjoining columns, beams, and braces should be concentric. However, the AISC341 provisions allow eccentricities less than the beam depth if the resulting member and connection forces are addressed in the design. No information is provided in EC8 related to the acceptable level of eccentricities of members. Although presumably no such eccentricities are allowed as per EC8, Astaneh-Asl [43] has shown that such relatively small amount of eccentricity, if introduced correctly, can improve the ductility of the gusset-plated connection without increasing the size of the gusset plate or the beam.

In both provisions, three broad geometries are defined for CBFs, namely, single diagonal or X-bracing (Fig. 1), V-bracing or inverted V-bracing (Fig. 2a–c), and K-bracing (Fig. 2d). The K-braced system is forbidden in both provisions for structures designed for seismic loading due to the possible column plastic hinging at the mid-height as a result of unbalanced brace forces. Among the diagonally-braced and V-braced systems, EC8 presents special cases (Figs. 1c and 2c) where the diagonals can be discontinuous. This type of a bracing is referred to as the y-bracing [44] which allows for larger openings. While EC8 recognizes this system as a viable option, no specific design requirements are given in the European provisions.

### 3.2. Seismic demands

To make a fair comparison, seismic demands (i.e. the load effects) should also be considered. In the United States, seismic demands on structures are calculated based on the regulations of the Minimum Design Loads for Buildings and Other Structures [45], hereafter referred to as ASCE7-10. In Europe, on the other hand, EC8 provisions are used for seismic loading. Both provisions define a design response spectrum to be used for determining the design base shear force.

In ASCE7-10, two spectral acceleration values, $S_s$ and $S_1$, are considered which are established using acceleration maps and depend on the location of the structure. The $S_s$ and $S_1$ parameters are based on risk targeted maximum considered earthquake ($MCE_R$) ground motions and are defined as mapped $MCE_R$, 5% damped, spectral response acceleration parameter at short periods and at a period of 1 s, respectively. These acceleration values are modified to arrive at $S_{MS}$ and $S_{M1}$ which are the $MCE_R$ spectral response acceleration parameters adjusted for site class effects. These parameters are finally multiplied by a factor of 2/3 to arrive at $S_{DS}$ and $S_{D1}$, which represent design spectral response acceleration parameters.

In EC8, the design response spectrum depends on a single acceleration parameter, $a_g$, which is the design ground acceleration on type A ground. This parameter together with the soil factor, $S$, are directly used in defining the design response spectrum. In the European approach two types of response spectra are defined, namely, Type 1 and Type 2. National Annexes can provide detailed information on which spectrum should be used. A comparison of the response spectra given by the American and European provisions for a high seismic region is given in Fig. 3. For the case of ASCE7-10, the values of $S_s$ and $S_1$ are considered to be equal to 1.5 g and 0.6 g, respectively. Furthermore, for a site class D (stiff soil), the design spectral accelerations, $S_{DS}$ and $S_{D1}$, are equal to 1.0 g and 0.6 g, respectively. For the EC8 spectrum, the value of $a_g$ is considered equal to 0.35 g and Type 1 spectrum is developed using ground type C. Fig. 3 shows that the response spectra developed based on ASCE7-10 and EC8 for a high seismic region with stiff soil are very close to each other.

In general, various methods are recommended in ASCE7-10 and EC8 for determining the earthquake-induced base shear and its height-wise distribution. The two most widely used methods are the equivalent lateral force procedure and the modal response spectrum analysis. Pushover and time history analysis procedures are also available, however, these are less frequently used in practice when compared to the former methods. Although the general principles are the same, there are

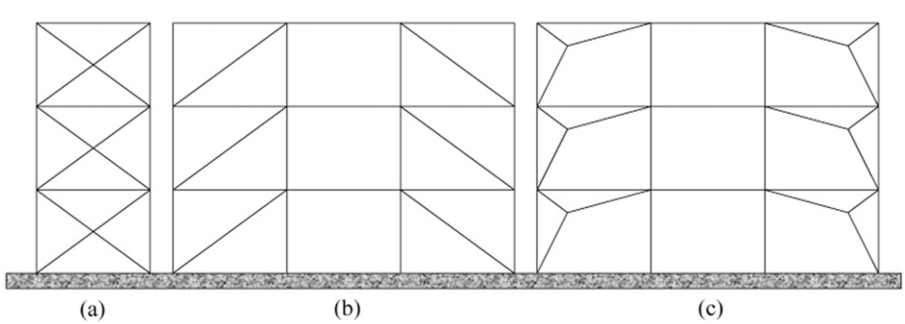


**Fig. 1.** Single diagonal and X-braced systems.