Soil Dynamics and Earthquake Engineering 113 (2018) 136–147

Contents lists available at ScienceDirect

Soil Dynamics and Earthquake Engineering

journal homepage: www.elsevier.com/locate/soildyn

The influence of moment resisting beam-to-column connections on seismic behavior of chevron concentrically braced frames

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ARTICLE INFO

Keywords:
Chevron concentrically braced frames
Eurocode 8
Bracings
Ductility
Seismic design
Beam-to-column joints

ABSTRACT

Chevron concentrically braced frames (C-CBFs) are very popular lateral-resisting systems, often opted for steel buildings due to their improved architectural functionality and reduced cost of fabrication and erection as X bracings. According to EN 1998-1 these systems are expected to provide limited ductility, thus penalizing the design of C-CBFs with smaller values of the behavior factor (namely $q = 2$ and $q = 2.5$ for medium "DCM" and high ductility class "DCH", respectively). To improve the ductility and the energy dissipation capacity of chevron bracings, the influence of the secondary frame effect provided by moment resisting beam-to-column connections belonging to the braced bays is investigated in this paper. To this aim, parametric non-linear analyses are carried out on a set of structural archetypes representative of low, medium and high-rise multi-story buildings. The results show that fully restrained joints can have beneficial effects providing an additional reserve of strength, stiffness and ductility.

1. Introduction

Chevron concentrically braced frames (C-CBFs), also known as inverted-V bracings, are very popular lateral-resisting systems, widely opted for steel buildings in seismic areas. Inverted V bracings are often preferred to X-CBFs since the location of openings (doors, windows) is easier and it requires fewer connections, as well as less material for the braces [1–3]. Moreover, for the values of interstory height and span length commonly used in low and medium-rise steel buildings, the chevron configuration guarantees appropriate slopes of the diagonal members (i.e. in the range $30^\circ \div 60^\circ$, as shown by [4]) that are suitable for the proper design of gusset plate connections.

Chevron bracings provide large lateral stiffness, which allows easily meeting both the drift limitations and the stability requirements. On the other hand, as widely observed in previous studies [5–15] such structural systems may exhibit relatively poor inelastic response under severe seismic ground motions.

The seismic performance demand of C-CBFs is strongly influenced by the type of developed plastic mechanism, which strictly depends on the flexural behavior of the brace-intersected beam. Indeed, if that beam experiences flexural yielding following the brace buckling, the structure can undergo significant loss of strength and stiffness and very poor energy dissipation capacity [5–7]. To prevent such detrimental behavior, current seismic codes [16–18] provide capacity design rules to assure "strong beam mechanism", which aims at enforcing the energy dissipation capacity in the braces under tension, while beams, columns and connections are kept in elastic range. Moreover, as highlighted by [5–8], even the beam flexural stiffness, beside its strength, should be controlled to guarantee an effective seismic response of chevron bracings. Indeed, large vertical deflection of the beam can prevent the yielding of brace under tension and impose severe ductility demand to the compression diagonal, thus leading to a very poor overall performance due to the brace deterioration.

Furthermore, both numerical and experimental evidences show that C-CBFs are prone to soft-story mechanisms in the most of cases. Several Authors [9–11,19–22] observed that the design rules currently codified in Eurocode 8 are less effective in assuring uniform distribution of plastic demand along the building height.

In the framework of EN 1998-1, chevron concentrically braced frames are expected to provide limited ductility and smaller behavior factors are recommended (i.e. $q = 2$ and $q = 2.5$ for medium "DCM" and high ductility class "DCH", respectively) as respect to X-CBFs (i.e. $q = 4$ for both DCM and DCH). On the contrary North-American seismic codes do not adopt such distinction [17,18], since these codes stipulate that the ductility class solely depends on the design requirements, and the behavior factor is assumed the same for both configurations.

Several researchers [7,8,21,23–28] proposed and investigated alternative design criteria to improve the seismic response of chevron bracings under severe ground motions. The outcomes of a recent comprehensive numerical parametric study [11] showed that the

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https://doi.org/10.1016/j.soildyn.2018.06.001
Received 6 April 2018; Received in revised form 19 May 2018; Accepted 2 June 2018
0267-7261/

design rules given by North American codes [17,18] are more effective than those recommended by the current version of EC8, which leads to the poorest energy dissipation capacity. In the light of such results, Costanzo et al. [23] proposed alternative design criteria to improve the seismic performance of medium ductility designed chevron bracings.

Further recent research findings concern the influence of the contribution to lateral strength and stiffness of the secondary action frame provided by the gusset plates and beam-to-column connections of the braced bays for both high [29–31] and moderate [32–35] seismicity.

In the US practice, moment-resisting (either full or semi rigid) beam-to-column joints into the braced bays are adopted for Special Concentrically braced frames SCBFs (namely for high ductility class). Uriz and Main [36] investigated by full scale test the contribution of beam-to-column gusset plate connections to the lateral load response of 2-story chevron special concentrically braced frame designed according to AISC341-10 [37]. Tests results showed that significant contribution was provided by the beam-to-column connections which resisted about 30% of the peak lateral load after the buckling of braces.

At current stage, Eurocode 8 does not specifically account for the secondary frame contribution and does not provide any requirement for beam-to-column joints belonging to the braced bays. On the contrary, Japanese Building Code [38,39] assigns the behavior factor even depending on the contribution of the secondary frame action: the larger is such contribution and the larger is the expected dissipation capacity.

In the light of these considerations, the research presented in this paper is addressed to investigate the possibility to furtherly improve the Eurocode 8 rules for chevron concentrically braced frames, by accounting for the extra strength and stiffness, as well as the ductility reserve, given by the secondary frame action, provided that beam-to-column joints belonging to the braced bays are designed to be fully moment restrained.

With this aim, a set of low, medium and high-rise frames equipped with chevron bracings were alternatively designed assuming either pinned or moment-resisting beam-to-column joints into the braced bays. Both nonlinear static and dynamic analyses were performed to evaluate and compare the seismic performance of examined cases.

The paper is organized in two main parts: the first part briefly summarizes the assumed design criteria, while the results of non-linear analyses are presented and discussed in the second part.

2. Design criteria

The investigated structures have been designed according to the criteria recently proposed by Costanzo et al. [23], formerly developed with the aim to revise the capacity design rules of the current EN1998-1 [16]. Furthermore, additional requirements for beam-to-column joints belonging to the braced bays are introduced in this study. For the sake of clarity, the adopted design assumptions are briefly summarized as follows:

- The seismic-induced effects on bracing members are calculated by performing a linear modal analysis considering both diagonals active in tension and compression.
- Cross-sectional Class 1 according to EN1993:1-1 [40] are selected for braces.
- The design resistances of the braces are assumed to satisfy the following conditions:

$$N_{b,br,Rd,i} \geq N_{Ed,br,i} = N_{Ed,br,E,i} + N_{Ed,br,G,i} \quad \text{at the } i\text{–th story} \tag{1}$$

$$N_{b,br,Rd,rf} \geq N_{Ed,br,rf} = N_{Ed,br,E,rf} \cdot q + N_{Ed,br,G,rf} \quad \text{at the roof} \tag{2}$$

- Where $N_{b,br,Rd,i}$ and $N_{b,br,Rd,rf}$ are the factored buckling capacity of the braces at the i-th and roof level, that are evaluated according to EN 1993:1-1 [40]; $N_{Ed,br}$ and $N_{Ed,br,rf}$ are the axial force acting in the bracing members at the i-th and roof level; $N_{Ed,br,E,i}$ and $N_{Ed,br,E,rf}$ are the axial force at the i-th and roof level due to the seismic action; $N_{Ed,br,G,i}$ and $N_{Ed,br,G,rf}$ are the axial force at the i-th and roof story due to the non-seismic actions included in the combination of actions for the seismic design situation. The condition expressed by Eq. (2) aims at keeping the bracings at the roof story in the elastic range to limit the damage concentration at the upper stories. Thus, the top story behaves similarly to an outrigger beam that modifies the shape of lateral displacements from cantilever-like to shear-type. The requirement expressed by Eq. (2) is disregarded in low rise frames, since stocky and short structures do not generally exhibit cantilever-type behavior. The EN 1998-1 limitation on the non-dimensional slenderness of bracing members (which should be less than or equal to 2.0) is retained.

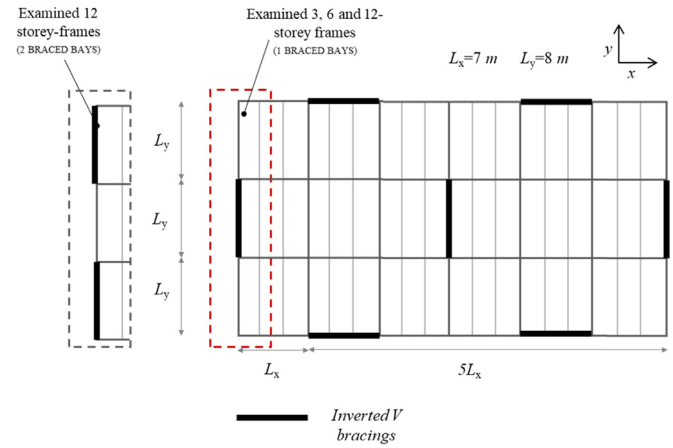

Fig. 1. Plan layout of the examined structural archetypes.

- To enforce a uniform sequence of buckling of braces and the corresponding variation of the post-buckling stiffness along the building height, the following condition is imposed:

$$[(\Omega_{b,i} - \Omega_b)/\Omega_b] \leq 0.25 \tag{3}$$

Where $\Omega_b = \min(\Omega_{b,i}) = \min\left(\frac{\chi \cdot N_{pl,br,Rd,i}}{N_{Ed,br,i}}\right)$ with $i \in [1, (n-1)]$ and $\Omega_{b,i}$ is capacity-to-demand ratio at the i-th story.

The requirement given by Eq. (3) differs from the EC8-compliant condition for the story-to-story variation of brace overstrength. Indeed, Eq. (3) aims at controlling the sequence of the buckling of the diagonal members, while according to EN1998-1 the designer should check the variation of the brace yield strength. However, the brace buckling under compression is the first nonlinear event. Once occurred, the story lateral stiffness drastically decreases, and the seismic demand tends concentrating in those stories. Hence, as demonstrated by [23], the compression-based approach to define the capacity-to-demand ratio allows obtaining almost uniform sequence of the buckling of braces and satisfactory distribution of lateral displacements along the building height, also reducing the tendency to soft-story mechanisms. Moreover, it is also worth noting that the requirement given by Eq. (3) simplifies the design process. Indeed, since the iterations to select the cross sections of diagonal members decreases, being easier to satisfy both strength checks and the slenderness requirements of the braces.

- The non-dissipative members (beams, columns and connections) are designed to withstand the most unfavorable condition between
  (1) the seismic-induced effects evaluated by means the former elastic analysis and magnified by tension overstrength factor $\Omega_y = \min\left(\frac{N_{pl,br,Rd,i}}{N_{Ed,br,i}}\right)$, without any requirements for its story-to-story variation;
  (2) the internal forces calculated performing a plastic mechanism