



Feasibility of Strongback System in Storey Mechanism Mitigation of Steel Braced Frames

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Abstract. Under earthquake loading, conventional braced frames are prone to storey mechanism. To mitigate this drawback, researchers have proposed to add an alternative vertical force path to redistribute member forces among floors. Likewise the dual system, the strongback acts as a vertical elastic spine and compensates for the loss of storey shear after braces of ductile system exhibited buckling. Nevertheless, the inelastic first mode and the elastic higher vibration modes excite the strongback, which could be installed interior or exterior to the braced frame. A design method for strongback braced frame is proposed and a case study consisting of a 4-storey office building in Victoria, BC, Canada, is presented. Numerical model was developed in the OpenSees environment and results from nonlinear response history analyses, expressed in terms of inter-storey drift, residual drift, and floor acceleration, are presented. The hybrid system is able to mitigate the storey mechanism and enhances the building's safety.

Keywords: Strongback system · Seismic response · Storey mechanism · Drift

1 Introduction

Conventional concentrically braced frames (CBF) are designed for ductility and the capacity design principle is applied to size the beam and column members to behave elastically under the design level earthquake. Nevertheless, the CBF system shows several drawbacks such as: i) reduced compression strength of buckled braces, ii) concentration of damage within a floor followed by soft-storey mechanism and iii) low redundancy. These vulnerabilities reduce the overall strength of CBF system, increase the repair cost, and reduce the system's redundancy after strong earthquake events. When residual drift increases above the $0.5\%h_s$ threshold, the building is irreparable and considered total economic loss [1]. To mitigate these drawbacks, structural systems such as: the zipper braced frame [2], dual system [3] and strongback/spinal braced frame [4–7] were proposed. Although the zipper braced frame was able to form the full-height zipper mechanism, the system is prone to dynamic instability after half-frame braces reached buckling almost simultaneously and beams hinged at their mid-span length. Then, the Dual system is able to improve the building's safety until the moment resisting frame (MRF) experienced beams hinging. Employing the Dual CBF-MRF, damage concentration is mitigated and residual drifts decreased.

The spine or strongback braced frames (SBF) are hybrid systems combining a stiff elastic spine (SB) with a conventional seismic force resisting system (SFRS). According to [4], the SB is an elastic truss that provides stiffness and strength to the overall system, prevents the formation of a weak-storey mechanism and promotes uniform drift distribution across the building height, while the ductile fuses of the counterpart system dissipate the input energy. The location of SB is either interior or offset to the ductile braced frame. The design procedure for the SBF is not straightforward. When the SBF is subjected to earthquake shaking, the SB is excited by the inelastic first mode and the elastic higher-modes; hence, its response is similar to the bending response of shear walls or rocking-braced frames [8].

In this paper, the seismic response of SBF with offset SB is analysed against that of MD-CBF (moderately ductile concentrically braced frame). The efficiency of design methodology in predicting the inelastic response of ductile braces and that of the elastic strongback truss of SBF system is demonstrated through a series of nonlinear dynamic analyses performed at design level and beyond.

2 Methodology

Several configurations of hybrid system are illustrated in Fig. 1. As depicted, the ductile system is the MD-CBF with either symmetric (Fig. 1a) or asymmetric (Fig. 1b) split-X braces, designed to behave in tension and compression. As shown, the SB of SBF could be installed either interior, exterior, or reversed exterior to the MD-CBF.

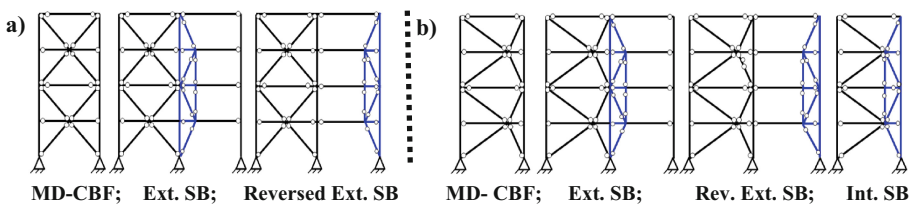


Fig. 1. Conventional MD-CBF and SBF systems: a) MD-CBF with symmetric split-X braces and the associated SBFs, b) MD-CBF with asymmetric split-X braces and the associated SBFs

Design methods for proportioning the spinal eccentrically braced frames and strongback braced frames were proposed by Chen et al. [6] and Simpson [7], respectively. These methods are based on the Modal Pushover Analysis (MPA) [9] and Modified Modal Pushover Analysis (MMPA) [10]. The MPA was developed for linearly elastic structural systems and is equivalent to the response spectrum analysis (RSA). The peak modal response resulted from the first mode (r_1) and higher modes (r_2, r_3, r_i) that were combined according to the square-root-of-sum-of-squares (SRSS) or the complete quadratic combination. The SRSS rule, which is valid for building structures with uncoupled modes, provides an estimate of the peak value of total response, r , as per Eq. (1).

$$r = \left(\sum_{i=1}^n r_n^2 \right)^{1/2} \tag{1}$$

Analysing a 9-storey MRF, it resulted that the 1st mode pushover analysis is able to identify damage at lower floors but is unable to detect damage in the upper floors, where the higher modes participation is required. According with the improved MMPA procedure [10], the seismic demands due to higher modes are computed considering the structural system elastic. Therefore, compute the response due to the first mode under the assumption that the system perform inelastically and the response due to higher modes under the assumption that the system remains elastic. Then, determine the total response by adding the gravity response, r_g , to the peak modal responses obtained using the SRSS combination rule as follows:

$$r = r_g + \left(\sum_{i=1}^n r_n^2 \right)^{1/2} \tag{2}$$

An adapted computation method developed to trigger forces in the controlled rocking steel braced frame system was proposed in [8]. Thus, the first mode response, r_1 , was obtained from the code prescribed lateral forces [11], and the elastic higher modes response (r_2 through r_M) was calculated as per the MPA. In [6], a similar procedure was developed but the elastic higher mode response was calculated as per the MMPA. Then, the frame member forces are combined using the modified SRSS combination as per Eq. (3). It is assumed that the spine or SB oscillates due to higher modes excitations, while the braced frame sustains the yielding of seismic ductile fuses experienced in the first-mode response. In addition, it is assumed that higher modes are uncoupled or weakly coupled. Herein, the SB system is designed according to Eq. (3) derived from the MMPA method, while in [7] it was derived from the MPA.

$$r = r_g + |r_{1,pl}| + \sqrt{r_{2,el}^2 + r_{3,el}^2 + \dots + r_{M,el}^2} \tag{3}$$

A graphical illustration of Eq. (3), adapted for SBF with offset SB, is shown in Fig. 2.

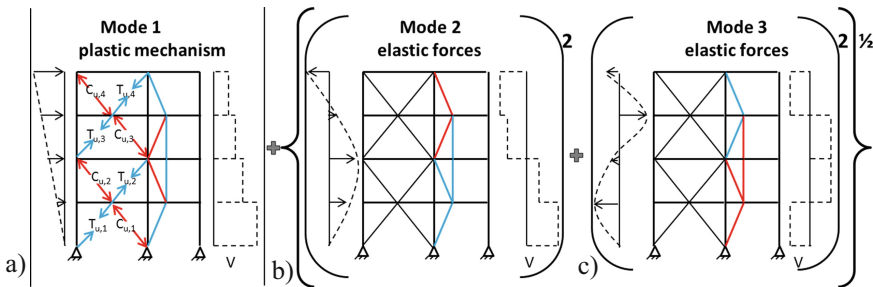


Fig. 2. Schematic MMPA procedure adapted as per Eq. (3) for SB members design of SBF

For the proposed design methodology, the steps to follow are:

1. Design the seismic ductile fuses (braces) of conventional braced frame by applying the Equivalent Static Force Procedure (ESFP) as per the building code [11]. Then, use the capacity design approach [12] to design the beams and columns of braced frame.
2. Determine the elastic natural periods and modes using an eigenvalue analysis.
3. Find the member forces assuming the structure forms a full plastic mechanism in the first mode (Fig. 2a). Then, use equilibrium of forces and deformation compatibility at each floor to compute the forces triggered in the SB as per the first inelastic mode.
4. Compute the equivalent-lateral force pattern for the 2nd and 3rd modes by RSA applied on 3D model, using the elastic truncated spectrum plotted in Fig. 3.
5. Calculate the forces in SB members resulted from the 2nd and 3rd elastic modal responses as per Figs. 2b and c.
6. Calculate the total response by combining the first and higher-mode response quantities using the SRSS combination according to Eq. (3) and Fig. 2.

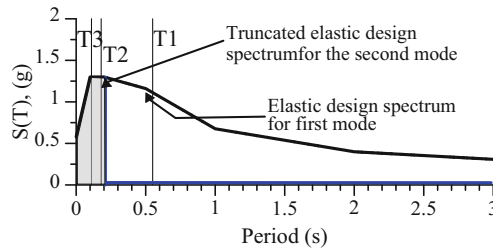


Fig. 3. Design spectrum according to building code and the truncated elastic design spectrum

3 Case Study

3.1 Design of Archetype Buildings

A 4-storey office building located on Site class C in Victoria, BC, is considered as case study and two types of SFRSs such as: i) the MD-CBF (Fig. 1a) and ii) SBF with offset SB (Fig. 1a) are employed. The floor plan and the numerical model for SBF with offset SB, associated with $\frac{1}{4}$ of building's floor plan, are shown in Fig. 4. The design of MD-CBF system is conducted according to building code [11] and steel design standard [12]. The ductility- and overstrength-related force modification factor are $R_d = 3$ and $R_0 = 1.3$, respectively. The spectral ordinates for 0, 0.1, 0.2, 0.5, 1.0 and 2.0 s are 0.58, 1.3, 1.3, 1.16, 0.676 and 0.399 g. According with ESFP [11], the base shear is calculated as $V = I_E S(T_a) W / R_d R_0$, where I_E is the importance factor ($I_E = 1.0$), $T_a = 0.05H$ is the fundamental period, H is the building height ($H = 14.8$ m), and W is the seismic weight. These parameters are provided in Table 1. All braces are made of hollow structural sections (HSS) and all beams and columns are I-shape sections. For HSS and I-shape members the yield strength, F_y is 350 MPa. In capacity design,

the probable yield strength of HSS braces should be not less than 460 MPa. All beams are pinned to the columns face and all braces to frame connections are pinned.

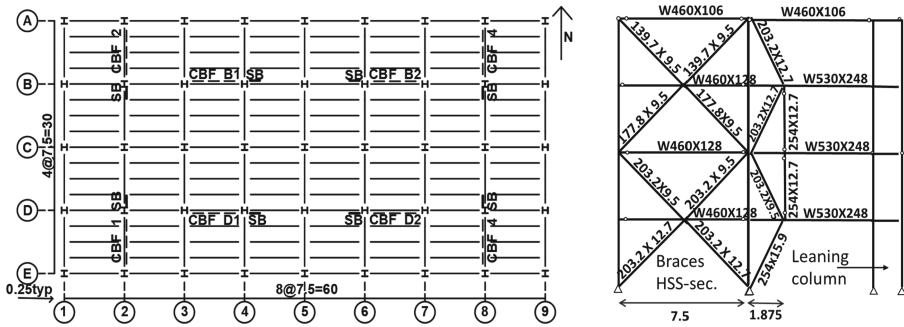


Fig. 4. Case study: floor plan and numerical model of SBF considering 1/4 floor area

From eigenvalue analysis conducted on the 3D building model using the commercial software ETABS, the 1st, 2nd, and 3rd mode periods in the direction of calculation (N-S) are also provided in Table 1. As resulted, T_1 is shorter than T_a , and each member's section was verified against forces obtained from the dynamic distribution. To design the SB (truss), the methodology described above is used. It was found that the stiffness of SB increases the total stiffness by only 6% and attention should be given when designing beams that support the offset SB's ties of SBF.

Table 1. Characteristics of archetype buildings

Struct. Typ.	ESFP			Linear dynamic analysis (ETABS)				Eigenvalue analysis (OpenSees)		
	T_a (s)	W (kN)	V (kN)	T_1 (s)	T_2 (s)	T_3 (s)	V (kN)	T_1 (s)	T_2 (s)	T_3 (s)
CBF	0.74	29457	6950	0.550	0.201	0.124	8476	0.554	0.205	0.138
CBF + SB	0.74	29457	6950	0.521	0.170	0.103	8995	0.523	0.176	0.111

3.2 Nonlinear Seismic Response

The seismic hazard for Victoria, BC, implies the consideration of crustal and subduction ground motions (GM) due to the proximity to Cascadia subduction zone. Crustal records were selected from Northridge ($M_w = 6.7$) and Loma Prieta ($M_w = 6.9$) earthquake events and the subduction records from the megathrust ($M_w = 9$) Tohoku earthquake in Japan (2011). To conduct nonlinear response history analyses, two suites of 7 GMs were selected and scaled to match the design spectrum computed for 2%/50 years probability of exceedance [11]. The response spectrum of individual records, their mean, and design spectrum are plotted in Figs. 5a and b for the crustal and

subduction GM suite, respectively. The numerical model shown in Fig. 4 is developed in OpenSees environment, using the same modelling techniques as those given in [13].

To assess the seismic response of MD-CBF building against that of SBF with offset SB, the recorded parameters are: the interstorey drift (ISD), residual interstorey drift (RISD), floor acceleration (FA), and the storey shear force. For comparison purposes, 2 crustal and 2 subduction GMs, scaled to design level (D.L.), were considered and the seismic response is presented in Fig. 6. As shown, the SBF is able to mitigate the peak ISD observed in the response of MD-CBF system under 3 out of 4 GMs (#739-250, #IBR004, #IBR006). When the building was subjected to GMs with high spectral acceleration ordinates at short periods (e.g. #IBR004 and #IBR006 in Fig. 5b), increased FA values were recorded. From the seismic response under #IBR004 GM, it resulted that the 3rd mode contribution to the SRSS combination led to larger FA demand at lower floors and higher drift at the top floor. In all the other cases, the 2nd mode governs the response.

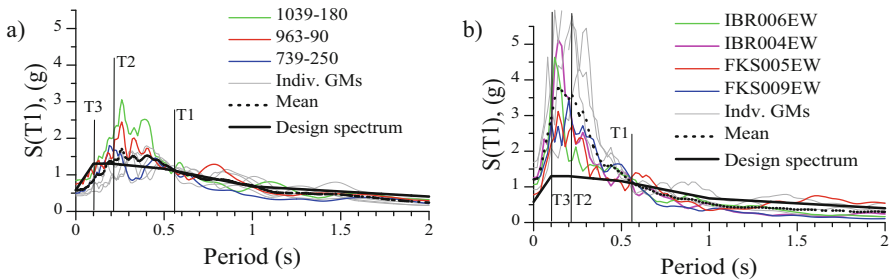


Fig. 5. Design spectrum for Victoria, BC, and scaled response acceleration spectrum of individual GMs: a) crustal GM suite and b) subduction GM suite

The hysteresis loops exhibited by all ductile braces of MD-CBF and SBF, as well as, the braces of SB truss under #1039-180 GM scaled at D.L. ($S(T_1) = 1.11g$) and at the intensity that triggered failure of MD-CBF ($S(T_1) = 2.3g \sim 2.09$ D.L.), are plotted in Fig. 7. From analyses, it resulted that at D.L., the left and right braces of top and bottom floor of MD-CBF showed inelastic response. When the demand increased to $S(T_1) = 2.3g$, both braces of 3rd floor of MD-CBF reached failure caused by low-cycle fatigue, while the top floor braces are in the verge of failure. Conversely, under the same ground motion scaled at D.L., the SBF follows the 1st mode response; hence, one brace of bottom and one of top floor experienced inelastic response, while the SB's members behave elastically. At increased demand that caused failure of MD-CBF, the SB is still in the elastic range. The SBF reached failure under increased intensity, $S(T_1) = 3.4g$, when both ductile braces (DB) of 1st floor have already exhibited failure caused by low-cycle fatigue, and one SB brace and one tie member experienced yielding, as illustrated in the IDA curve plotted in Fig. 8a. Examining the behaviour of SBF under earthquake shaking, it was observed that RISD substantially decreased which means less economic loss and reduced repair time (e.g. replacement of damaged DBs). A similar response of SBF was observed under #739-250 GM.

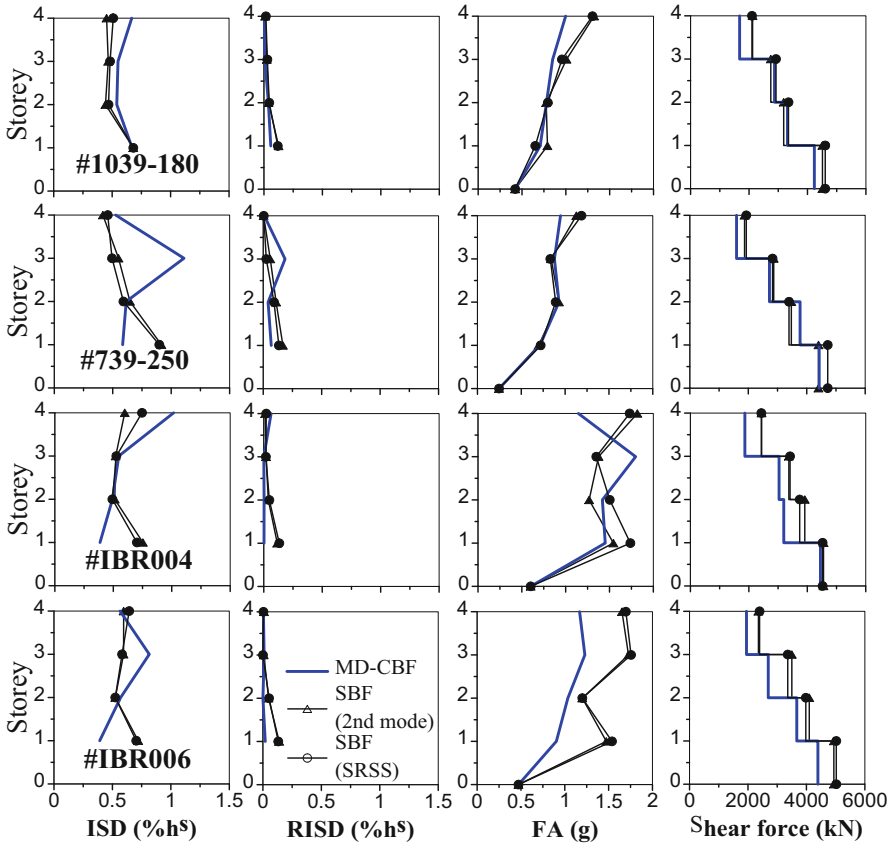


Fig. 6. Response of MD-CBF vs. SBF in terms of ISD, RISD, FA, and storey shear (D.L.) under GMs: crustal #1039-180, #739-250 and subduction #IBR004, #IBR006

However, under #963-90 GM, the failure of second DB occurred after the yielding of one SB’s brace. Similarly, under 2 out of 3 subduction records (Fig. 8b), the failed DBs are at the 1st or 2nd floor and occurred prior to yielding of one SB brace and/or tie member. Conversely, when the failure of two DBs is recorded at top floor, as resulted under #IBR004, the yielding of SB’s brace occurred before the failure of DBs. It is worth mentioning that the RISD associated with the failure of two DBs is around 0.5% h_s , which assures the building reparability threshold after a seismic event. The presented design methodology is for low-rise buildings. For taller buildings, for which the higher modes contribute more, the segmental strongback system is recommended.

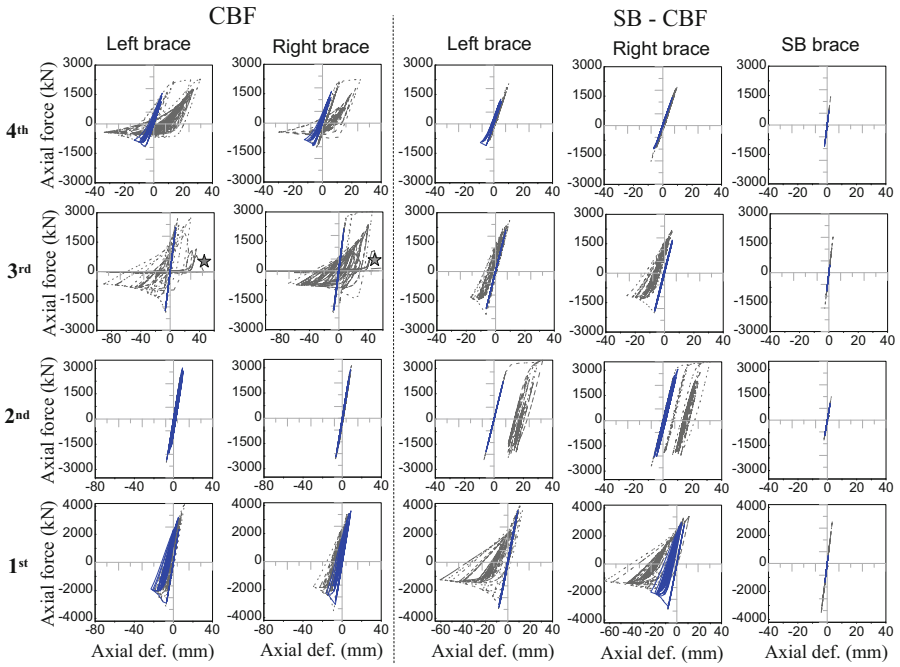


Fig. 7. Hysteresis loops of all braces of CBFs vs. SB-CBF under crustal GM #1039-180 scaled for code-based D.L. (blue) and 2.09 D.L. (grey) that triggered failure of 3rd floor CBF's braces

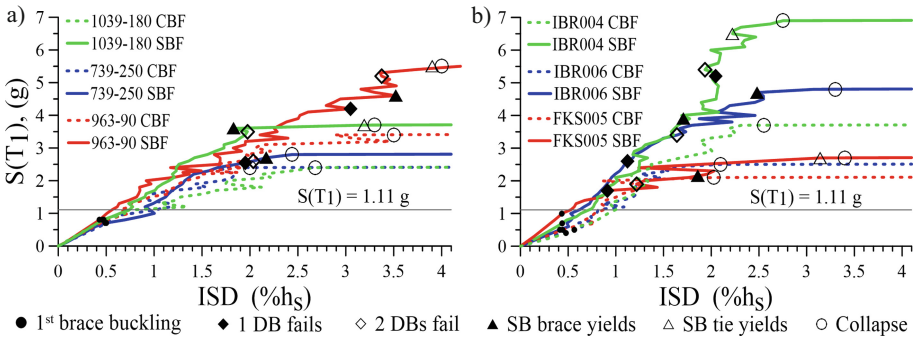


Fig. 8. IDA curves under a) crustal ground motions and b) subduction ground motions

4 Conclusions

This paper presents the seismic response of SBF system against the counterpart MD-CBF. A design methodology for the SBF is proposed and from nonlinear response history analysis including IDA is demonstrated the system's efficiency to mitigate the storey mechanism. As shown, the spine is able to distribute the demand across the

building's height, while the RISD is about $0.5\%h_s$, even though two ductile braces experienced failure caused by low-cycle fatigue. Thus, the main benefit of hybrid system is the improvement of redundancy and diminishing the economic loss by ideally replacing only the damaged ductile braces in the aftermath of seismic events.

References

1. McCormick J, Aburano H, Ikenaga M, Nakashima M (2008) Permissible residual deformation levels for building structures considering both safety and human elements. In: 14th WCEE, Beijing, China, Paper ID 05 06-0071
2. Tirca L, Chen L (2012) The influence of lateral load patterns on the seismic design of zipper braced frames. *Eng Struct* 40:536–555
3. Kiggins S, Uang CM (2006) Reducing residual drift of buckling-restrained braced frames as a dual system. *Eng Struct* 28:1525–1532
4. Lai J, Mahin SA (2014) Strongback system: a way to reduce damage concentration in steel-braced frames. *J Struct Eng* 141(9):1–11
5. Simpson BG, Mahin SA (2017) Experimental and numerical investigation of strongback braced frame system to mitigate weak story behavior. *J Struct Eng* 49(14):1–14
6. Chen L, Tremblay R, Tirca L (2019) Practical seismic design procedure for steel braced frames with segmental elastic spines. *J Construct Steel Res* 153:395–415
7. Simpson BG (2020) Higher-mode force response in multi-story strongback-braced frames. *Earthq Eng Struct Dyn* 49(14):1406–1427
8. Steele T, Wiebe L (2016) Dynamic and equivalent static procedures for capacity design of controlled rocking steel braced frames. *Earthq Eng Struct Dyn* 45:2349–2369
9. Chopra A, Goel R (2002) A modal pushover analysis procedure for estimating seismic demands for buildings. *Earthq Eng Struct Dyn* 31:561–582
10. Chopra A, Goel R, Chintanapakdee C (2004) Evaluation of modified MPA procedure assuming higher modes as elastic to estimate seismic demands. *Earthq Spectra* 3:757–778
11. National Research Council of Canada (2015) National Building Code of Canada (NBC)
12. Canadian Standards Assoc. (CSA) (2014) Design of steel structures, S16-14 standard, On
13. Tirca L, Chen L, Tremblay R (2015) Assessing collapse safety of CBF buildings subjected to crustal and subduction earthquakes. *J Construct Steel Res* 115:47–61