Canadian Conference - Pacific Conference on Earthquake Engineering 2023
Vancouver, British Columbia
June 25th – June 30th, 2023

Performance of Steel Frames with Buckling-Restrained Braces Subjected to Ground Motions

Emad Abraik[1], Asif Iqbal[1*]

[1] University of Northern British Columbia, Prince George, BC, Canada, V2N 4Z9

[*] asif.iqbal@unbc.ca

ABSTRACT

Buckling-restrained braces (BRB) are regularly used in steel buildings to resist lateral loads. Recent research has suggested biaxial and three-dimensional excitation cause high vibrations that lead to structural and non-structural component damage. In this paper a three-story building model subjected to horizontal (uniaxial and biaxial) and three-dimensional ground motions is investigated for responses of steel frames with BRB. The building is in a high seismic zone and designed in accordance with current Canadian National Building Code and Steel Code. Extensive numerical analyses with a range of ground motions are carried out to examine nonlinear seismic performance. Floor responses along building height are presented and compared with code limit.

Keywords: Buckling Restrained Braces; 3D-Ground shaking; Floor Accelerations

INTRODUCTION

Buckling Restrained Brace (BRB) frames are an innovative seismic-resistant system that provides ductility and stability to structures during earthquakes. The superiority of BRB over traditional braces is due to their capacity to withstand and provide support under both tension and compression forces [1] [2]. Although BRB braces effectively reduced horizontal shaking, potential damage to non-structural components resulted in significant economic losses.

A number of recent earthquakes have highlighted the importance of preventing damage to non-structural components and contents of buildings. The estimating of non-structural element vulnerability caused by peak horizontal floor acceleration was studied by Chaudhuri and Hutchinson [14], suggesting modifying the overstrength to reach a reliable estimate of the vulnerability of non-structural components. Merino et al. [15] also studied the impact of floor acceleration on the seismic performance of industrial structures supported with braced frames. Results were used to assess such structures as an uncoupled system under various damping ratios. However, the vertical ground motion components have been ignored in these studies.

Several researchers have turned their attention to investigating the influence of vertical ground motion on the seismic behaviour of structures equipped with BRB braces. Sultana and Youssef [3] concluded that the vertical seismic component intensified the axial column forces and the beams' vertical deformations, increasing the seismic damage state. This large deformation is favored by the vertical seismic component, especially when the loss of a column causes the structure's vertical integrity to deteriorating [6]. Similarly, Liu et al. [5] proposed a design approach to cover the influence of horizontal and vertical ground motion components on the seismic performance of six brace configurations. The BRB innovative design approach extends the role of bracing systems, from protecting structures from lateral loading to preventing vertical collapse. Although this research drew attention to the importance of vertical ground motion on the seismic performance of BRB structures, these studies have yet to consider the impact of all three components of ground motion.

To estimate the design force induced on the non-structural components, current seismic design provisions (e.g., ASCE 2016) [7] recommend a linear variation of acceleration along the height of the building. The ASCE 2016 provision's design philosophy seeks to ensure that these components are designed to withstand the design earthquake load without collapsing, toppling, or shifting. The ASCE 2016 code recommendations assume a trapezoidal acceleration distribution when calculating forces applied to non-structural components. Nevertheless, the ASCE 2016 [7] provisions do not explicitly account for the impact of vertical ground motion on non-structural components.

This paper compared the estimated floor acceleration using the approximate ASCE 7 [7] formula with the floor response spectrum of BRB buildings. The influence of vertical ground motion on BRB buildings' non-structural components when subjected to uniaxial, biaxial, and triaxial seismic excitation is considered.

STUDIED BUILDING

A three-story steel office building, shown in Figure 1, is analyzed to compare the impact of one-directional, two-directional, and three-dimensional ground motions on its peak floor acceleration (PFA) and floor response spectrum (FRS). The building is situated in Vancouver, BC, and is built on site class D with shear wave velocity values ranging from 180 m/s^2 to 360 m/s^2. The primary lateral support system in both directions consists of one bay of buckling restrained braces, while the remaining columns are designed as gravity columns.

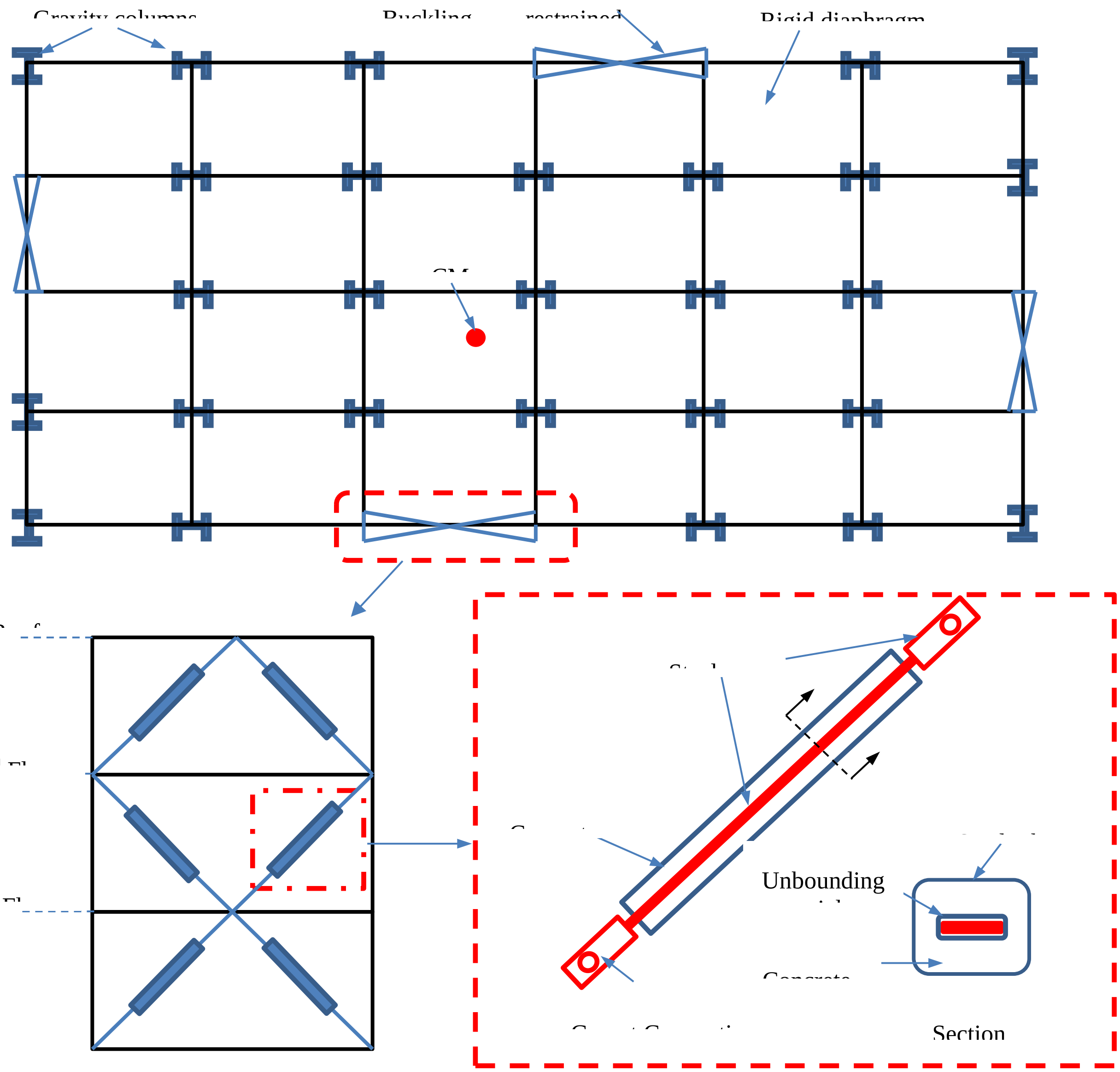


Figure 1. Building outlines and brace details

As per the NBCC [8] guidelines, the center of mass on each floor has been offset by 10% from the center of geometry to account for accidental torsional effects. The building was designed according to S-16 [9], assuming R_d R_o equals to 4.0 and 1.2, respectively for BRB frame. Table 1 lists the cross-section area of the brace members, gravity columns, and beams.

Table1. Cross-section area of building members

Elevation	Braces Frame			Gravity Beams (mm^2)	Gravity Columns (mm^2)
	Braces area (mm^2)	Beams (mm^2)	Columns (mm^2)		
Roof	8710	14387	12323	24710	70322
2nd floor	13548	14387	22710	34580	94838
1st floor	16581	14387	22710	34580	94838

NUMERICAL MODEL AND GROUND MOTIONS SCALING

A 3D- numerical model for the selected building was developed in the Open System for Earthquake Engineering Simulation software (OpenSees) [10]. The gravity beams and columns are modeled using nonlinear beam column element with fiber section. The braces are modeled using corotational truss element while the connected gusset plated are modeled using a rigid element. Rigid diaphragm is assigned at each story with 10% mass eccentricity. With a design utilization ratio ranging from 0.42 to 0.86, the calculated mass of the building is changed by increasing or decreasing it, resulting in different structural periods. Table 2 lists the structure periods and their corresponding mass. This allows to get produce different structures periods, and thus different floor response. The ground motions were applied a uniaxial (i.e., x-direction), biaxial (i.e., x and y-directions), and 3D (i.e., x, y, and z direction).

Table2. Mass variation and corresponding periods

Building	Seismic weight (kN)/story	T_1 (sec)	T_2 (sec)	T_3 (sec)
B1	2816	0.36	0.35	0.32
B2	8558	0.59	0.57	0.32
B3	16552	0.86	0.75	0.33
B4	21703	0.96	0.92	0.34

From a PEER ground motion database [11], 22 motions were chosen to match the expected maximum earthquake spectrum for Vancouver, BC. The Mean square error was used to adjust the scale of these motions, and their median was compared to the target design spectrum shown in Figure 2. This vertical target spectrum was determined using FEMA-P750 [12] approach, which involved deriving a vertical spectrum from a horizontal spectrum. The horizontal spectrum shall be modified as follows:

1- $\mathbf{0.3C_v S_{DS}}$ **for** $\boldsymbol{T \leq 0.025\ sec}$
2- $\mathbf{0.8C_v S_{DS}}$ **for** $\boldsymbol{0.05 < T \leq 0.15\ sec}$
3- $\mathbf{0.8C_v S_{DS}(0.15/T)^{0.75}}$ **for** $\boldsymbol{0.15 < T \leq 4}$

Where C_v varies from 0.7 to 1.5 based on the variation of S_{DS}. The peak ground acceleration (PGA) for each ground motion is shown in Figure 3.

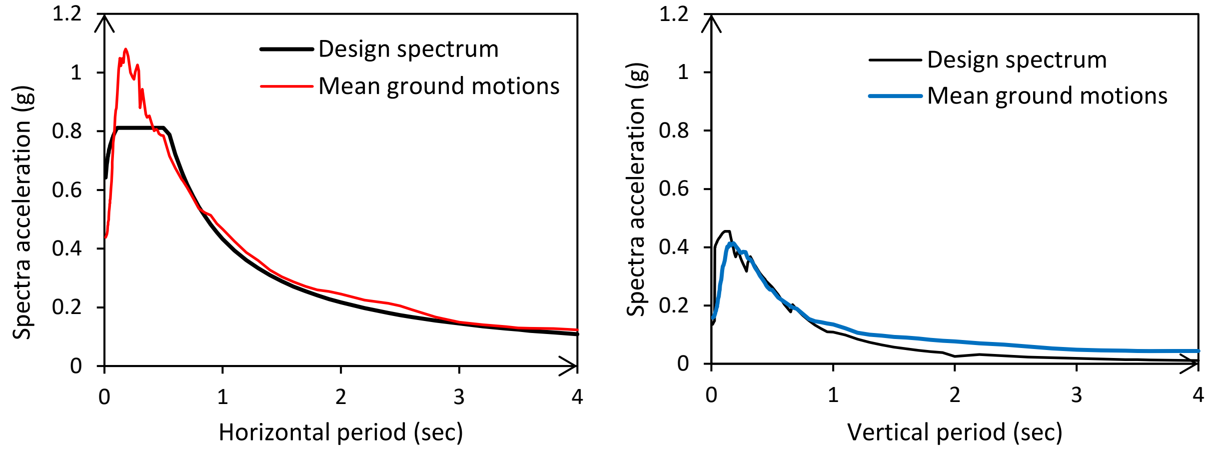


Figure 2. Acceleration response spectra for 22 ground motions, mean spectrum for: (a) horizontal direction; (b) vertical direction.

RESULTS

Floor response spectrum (FRS)

Floor responses can be accurately derived from a time history analysis. An accurate probabilistic view of FRS may require a sufficient number of tri-directional earthquake time histories [13]. Figure 3 shows the FRS response for the studied buildings. In the case of the flexible building B4, it can be observed that the peak floor acceleration manifests at a low period, followed by another peak acceleration occurring at a higher period. However, reducing the building's flexibility causes a shift in the behavior of the structure, resulting in the second peak becoming the highest and occurring at a lower period. This shift in peak acceleration behavior can have implications for the seismic performance of the building, which must be carefully considered and analyzed during the design and retrofitting processes.

In the case of a flexible building such as B4, the peak floor acceleration occurs at the first story, which corresponds to the first peak of the building's response spectrum. However, for a rigid building, this behavior can be significantly revised, with the peak acceleration occurring at the roof level and corresponding to the second peak of the response spectrum.

Compared to a flexible building, rigid building B1 exhibits less triaxial excitation influence. In contrast, building B2, which is more flexible, responds differently to triaxial excitation due to its increased flexibility.

Increasing the flexibility of buildings such as B3 and B4 does not seem to affect the difference in response between triaxial and uniaxial excitations. A careful analysis and design are essential for seismic-resistant building construction because of the complex interaction between building flexibility and triaxial excitation.

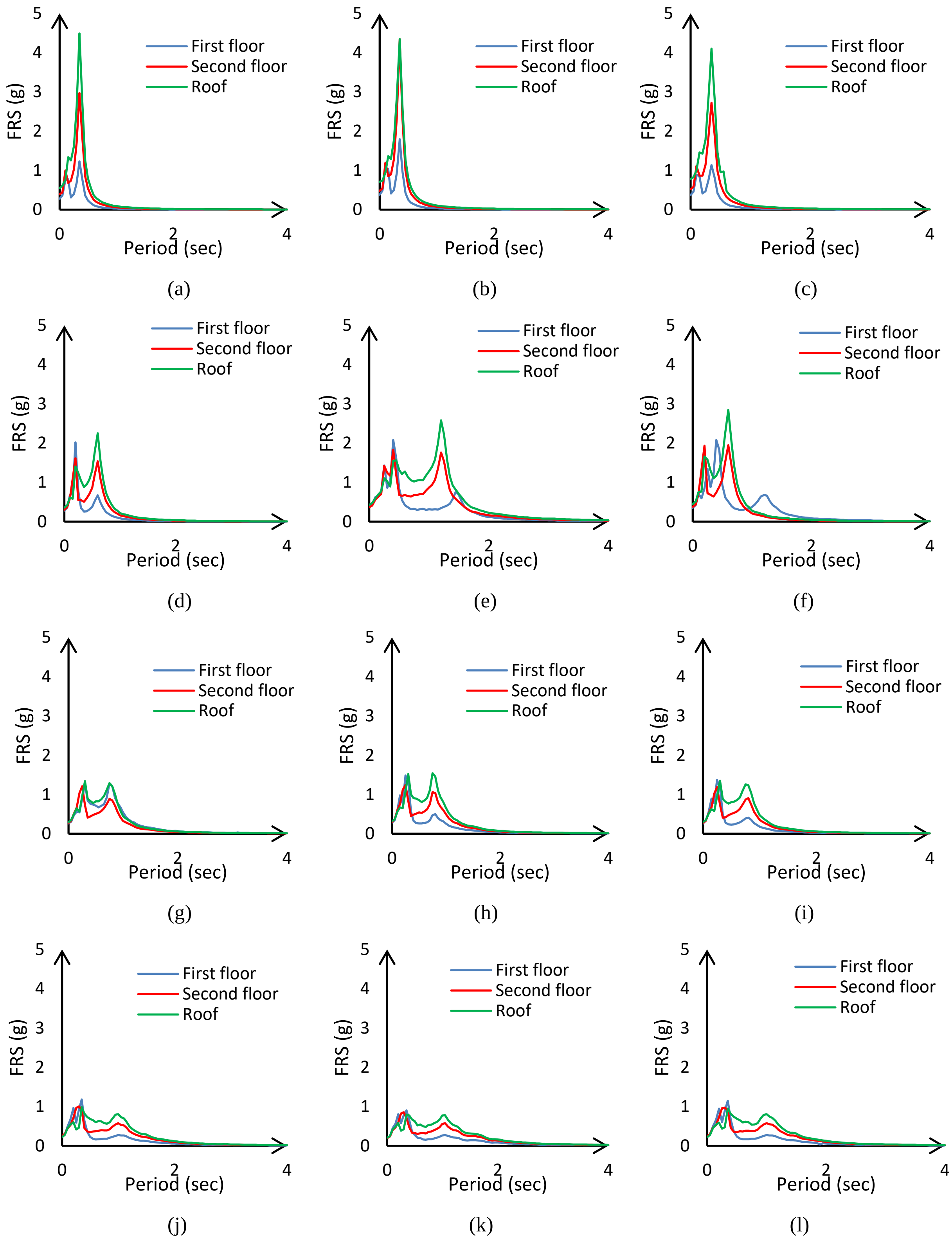


Figure 3. FRS in X-direction for: (a) B1 (uniaxial); (b) B1 (biaxial); (c) B1 (triaxial); (d) B2 (uniaxial); (e) B2 (biaxial); (f) B2 (triaxial); (g) B3 (uniaxial); (h) B3 (biaxial); (i) B3 (triaxial); (j) B4 (uniaxial); (k) B4 (biaxial); (l) B4 (triaxial)

PEAK FLOOR ACCELERATIONS (PFA)

Figure 4 plots the peak floor acceleration at the CM. The allowable floor acceleration (a_f) can be calculated using Eq (1) [7]:

$$\boldsymbol{a_f = \frac{0.4S_{Ds}}{I}\left(1+\frac{2z}{h}\right)} \quad (1)$$

where S_{DS} is the design spectral response acceleration parameter at short periods; z is the height of the component measured from the base, and the h is the roof height of the structure measured from the base.

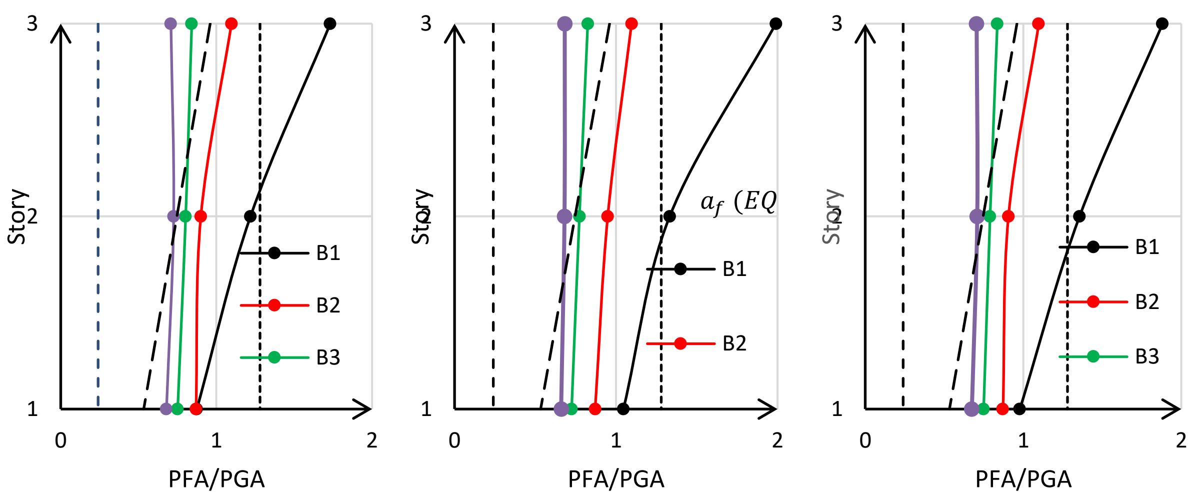
$a_f\ (EQ-1)$

It is a requirement that the upp $a_f(EQ-1)$ 1) shall not exceed $1.6S_{DS}$, while the lower limit shall not be l Figure 4 shows the computer floor acceleration, upper limit, lower limit, and average peak floor response spectra calculated from nonlinear time history analysis.

In EQ-1, the period of vibration of the non-structural component is not explicitly required nor are the periods of vibration of the building structure. Thus, non-structural components are prescribed as being flexible or rigid, as indicated by the assigned value of the component response amplification factor. The variation in the floor accelerations for the building are assumed to be linearly increasing over the height of the building, which is based on the dynamic behavior of the building response being predominantly due to the first mode. The non-structural force also does not account for differences in building behavior in different orthogonal directions.

Under different ground motion excitations, the forces computed using EQ-1 does not provide conservative estimation at first floor for all selected buildings. The upper limit of Eq-1, however, provides a conservative estimation except for B1 (i.e., rigid building). The influence of utilizing different ground motion components appear to have no impact on the estimated non-structural components design forces.

As can be seen in Figure 5, there is a clear distinction between the two methodologies. The dissimilarity between PFA and FRS appears to be significantly greater for the inflexible structure, as seen in the case of B1, and diminishes for the flexible structure, as demonstrated in the case of B4. For flexural buildings, as with B4, the effects of 3D ground motions appear to be limited.



(a) (b) (c)

Figure 4. Peak floor acceleration: (a) uniaxial; (b) bidirectional; (c) triaxial

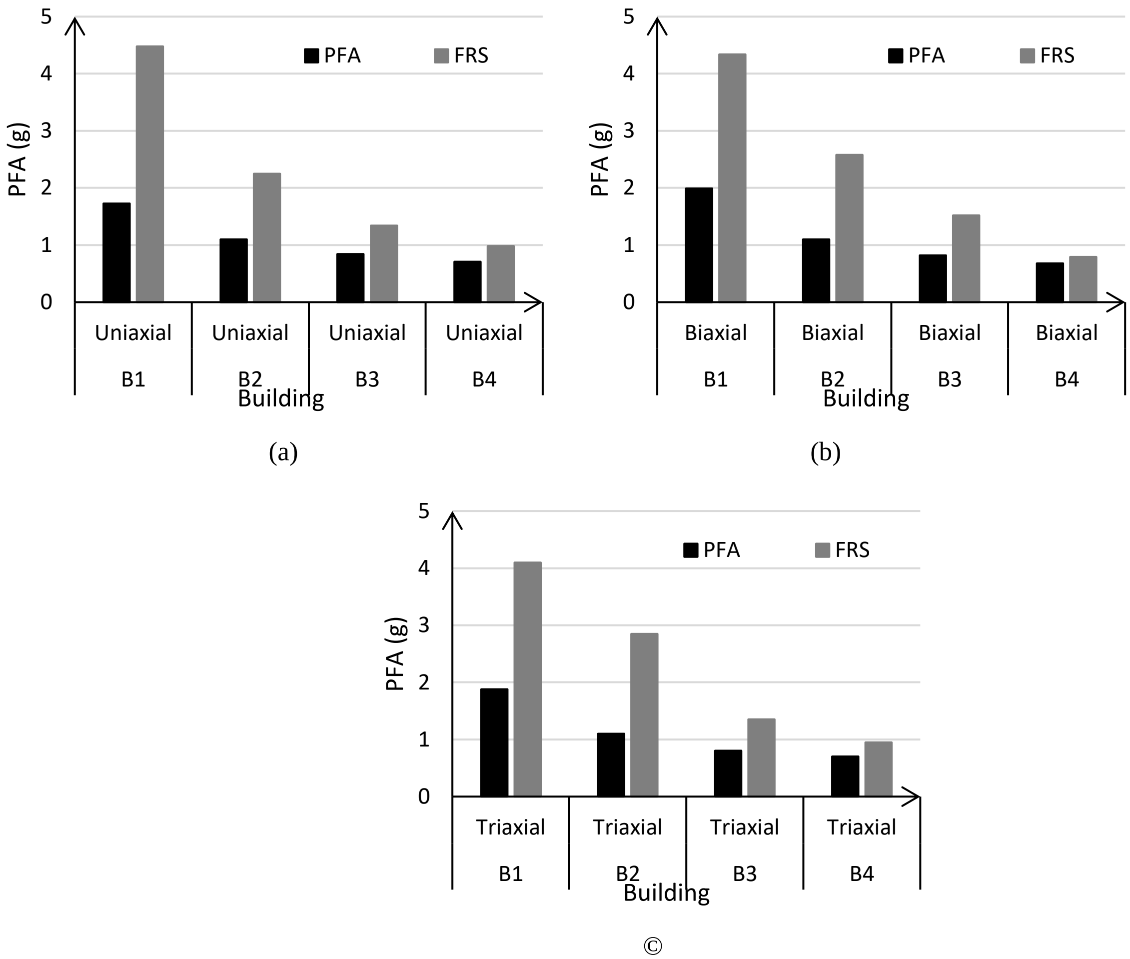


Figure 5. Comparison between peak floor acceleration and peak floor response spectrum for: (a) uniaxial; (b) bidirectional; (c) triaxial

CONCLUSIONS

This paper presents a comparison of the seismic behavior of a building equipped with Buckling Restrained Braces (BRBs) subjected to various ground motion components. A numerical model was developed to investigate the seismic response of a 3-story office building situated in a highly active seismic region in Canada. Based on the research conducted, the following conclusions were drawn:

1- Triaxial excitation is less pronounced in rigid building than it is in flexible building. In contrast, increased flexibility results in a different response to triaxial excitation.
2- To construct structures that can withstand seismic activity, it is necessary to combine the flexibility of the building with triaxial excitation as well as meticulous analysis and design.
3- Triaxial excitation is less pronounced in rigid building than it is in flexible building. In contrast, increased flexibility results in a different response to triaxial excitation.
4- A PFA method for estimating non-structural constituent forces is comparatively less accurate than a FRS method for estimating structural forces

Eventually, although this study highlighted the in the importance of considering the triaxial excitation on the floor response spectra response, other building locations are needed to be considered to fully understanding the behaviour. Moreover, the torsional eccentricity of the studied buildings was lumped and shifted 10%. Therefore, the variation on the mass eccentricity and its influence is needed to be determine in the future research work.

REFERENCES

[1] Jia, M., He, J., & Lu, D. (2022, July). Experimental research of seismic performance of buckling-restrained braced frame with ductile connections. In Structures (Vol. 41, pp. 908-924). Elsevier.

[2] Gong, C., Jia, M., Cheng, X., & Gritcuk, M. (2022, March). Seismic performance of buckling-restrained braced steel frames subjected to earthquake main shock-aftershock sequences. In Journal of Physics: Conference Series (Vol. 2230, No. 1, p. 012026). IOP Publishing.

[3] SULTANA, P., & YOUSSEF, M. A. (2015). Variation of Maximum Inter-Storey Drift for Steel Moment Resisting Frames Considering the Horizontal and Vertical Seismic Components. Response of Structures Under Extreme Loading (PROTECT 2015).

[4] Di Sarno, L., Elnashai, A. S., & Manfredi, G. (2011). Assessment of RC columns subjected to horizontal and vertical ground motions recorded during the 2009 L'Aquila (Italy) earthquake. Engineering structures, 33(5), 1514-1535.

[5] Liu, T., Tantely, J. S., & He, Z. (2022, April). Effect of vertical-to-horizontal acceleration ratio of earthquake components on the utilization of buckling-restrained braces in steel frames. In Structures (Vol. 38, pp. 1111-1124). Elsevier.

[6] Vela, R. M., Brunesi, E., & Nascimbene, R. (2018). Derivation of floor acceleration spectra for an industrial liquid tank supporting structure with braced frame systems. Engineering Structures, 171, 105-122.

[7] ASCE. Minimum design loads for buildings and other structures. ASCE/SEI 7-16 including Supplement No.1, Reston, VA.

[8] National Research Council of Canada. (2015). National Building Code of Canada 2015. Ottawa, ON: National Research Council of Canada.

[9] Canadian Standards Association. (2019). CSA S16-19: Design of steel structures. Rexdale, ON: Canadian Standards Association.

[10] McKenna F, Fenves G, Scott M. Computer program OpenSees: Open system for earthquake engineering simulation. Berkeley, CA: Pacific Earthquake Engineering Center, Univ. of California; 2013.

[11] Pacific Earthquake Engineering Research Center (PEER). Guidelines for the performance-based design of tall buildings, PEER Report 2017/06, University of California, Berkeley, CA.

[12] National Earthquake Hazards Reduction Program (NEHRP) (2009) Recommended Seismic Provisions for New Buildings and Other Structures (FEMA P-750). NEHRP.

[13] Villaverde, R. (1997). Seismic design of secondary structures: state of the art. Journal of structural engineering, 123(8), 1011-1019.

[14] Chaudhuri, S. R., & Hutchinson, T. C. (2004, August). Distribution of peak horizontal floor acceleration for estimating nonstructural element vulnerability. In 13th World Conference on Earthquake Engineering.

[15] Merino Vela, R. J., Brunesi, E., & Nascimbene, R. (2019). Floor spectra estimates for an industrial special concentrically braced frame structure. Journal of Pressure Vessel Technology, 141(1).