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THE IMPACT OF THREE-DIMENSIONAL MODELLING AND BIAXIAL SEISMIC EXCITATION ON THE NONLINEAR RESPONSE OF BRACED FRAME BUILDINGS

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Abstract

Post-earthquake investigations have revealed that torsion-induced damage is a significant contributor to building collapses, particularly in the case of flexible steel buildings characterized as torsionally-sensitive. Due to the complexity of three-dimensional nonlinear models, the number of available studies on torsional effect on steel braced frame buildings is limited. Meanwhile, the magnitude of accidental torsion depends on the expression of accidental eccentricity, which is different in the current National Building Code of Canada (NBC) versus the ASCE/SEI 7 standard. Hence, the latter yields larger accidental torsional moments for torsionally-sensitive buildings and lower for regular buildings. On the other hand, more studies released have focused on the understanding of the influence of biaxial excitation on the behaviour of buildings. Based on comparative nonlinear responses of regular and torsionally-sensitive buildings under both uniaxial and biaxial excitation, a number of studies showed the importance of considering biaxial excitation in the assessment of nonlinear seismic responses of buildings. This effect is more critical for the assessment of existing buildings that were designed for lower seismic intensity. The case study is a regular and torsionally-sensitive 8-storey MD-CBF office building in Montreal.

The objectives of this study are: (i) evaluate the accidental eccentricity expression provided in the current building code, (ii) highlight the effect of biaxial excitation vs. uniaxial excitation on the building seismic response, and (iii) assess the vulnerability of a middle-rise regular CBF office buildings designed and constructed in 1980s.

Keywords: Steel structures; Earthquake; Torsion effects; Nonlinear dynamic analysis

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1 INTRODUCTION

Irregular buildings are more vulnerable to earthquakes than their regular counterparts, particularly when subjected to biaxial ground motions (GMs). The NBC (2020) [1] classifies torsionally-sensitive structures as irregular and mandates dynamic analysis. Torsional effects arise from inherent and accidental eccentricity that significantly increase seismic forces and drift demands. There is a variety of accidental eccentricity (e_a) expressions among building codes. Thus, in [1], $e_a = \pm 0.1L$ but for regular buildings subjected to dynamic analysis the value diminished to $\pm 0.05L$, where L is the building dimension perpendicular to the direction of applied seismic load. In the ASCE/SEI 7-22 [2], $e_a = \pm 0.05L$ and for torsionally-sensitive buildings the torsional moment at each floor is amplified by a torsional amplification factor A_x. It is noted that in Eurocode 8 [3] the accidental eccentricity is $\pm 0.05L$. However, it is noted that accidental torsional moments induced in buildings with rigid diaphragm does not occur simultaneously at all floors.

To account for the effect of biaxial excitation, the "30% rule" proposed in [1] is generally used in seismic design. This rule suggests that the maximum effect of an earthquake on a building structure occurs when 100% of the seismic force is applied in one direction and 30% in the orthogonal direction. However, researchers pointed out that this rule can adequately predict seismic demand in the elastic range, but tends to underestimate the seismic demand in the post-elastic range [4]. Further studies conducted by researchers [5, 6] highlighted that bidirectional excitation amplifies drifts, particularly at flexible edges and leads to underestimated responses when biaxial effects are ignored. Furthermore, in [4] it was emphasized the necessity of considering GMs in both orthogonal directions, as the single-direction analysis fails to capture critical inelastic response.

Despite the advancements in modelling, research on torsion-induced effects in steel-braced frame buildings under biaxial excitation remains limited. To capture these effects, detailed three-dimensional (3D) nonlinear models are required, as simplified planar analyses fail to represent the coupled lateral-torsional behaviour induced by bidirectional GMs. This study addresses this gap by evaluating the collapse capacity of steel-braced frame buildings, quantifying the impact of accidental torsion, and assessing bidirectional seismic effects through detailed 3D modelling. It is noted that biaxial excitation amplifies torsional responses by inducing coupled lateral-torsional motions which lead to increased structural damage.

In [7] is shown that lateral load as wind also induces shear and torsion in buildings. The wind load becomes critical especially for taller buildings. A comparison between the effect of torsion induced by wind and earthquake will be shown in an extended paper.

It is worth mentioning that in Canada, the largest building stock was built in the 1980s [8] according with the 1980 edition of NBC [9]. Among these buildings, a large stock is represented by low- and middle-rise steel structures designed as per CSA/S16.1-M78 [10] standard. A seismic assessment of a regular prototype building designed in 1980 is presented herein.

Thus, the objectives of this study are: (1) evaluate the effect of accidental torsional moments in the design of regular and torsionally-sensitive buildings using 3D analyses, (2) highlight the effect of biaxial vs. uniaxial excitation on the building seismic response using 3D models, and (3) assess the seismic vulnerability of a middle-rise regular CBF office buildings designed and constructed in 1980s.

2 METHODOLOGY

To reach the above objectives, a detailed three-dimensional OpenSees model is developed and the seismic response of a torsionally-sensitive steel braced frame building (TSB) is discussed against its counterpart regular building (RB). The target of this study is to assess the seismic response of TSBs with and without torsion consideration, as well as when shear caused by torsion is computed in accordance with NBC 2020 [1] versus the ASCE/SEI 7-22 standard [2]. The steel tonnage associated with each design variant is provided. In addition, the effect of uniaxial versus biaxial excitation is also presented. A discussion about vulnerability of existing steel buildings in function of construction year and the associated seismic hazard is also provided.

3 CASE STUDY

The case study is a prototype 8-storey office building located on Site Class C (stiff soil), in Montreal, QC, Canada. The building has regular elevation and features a regular square footprint plan of 38.5m x 38.5m, as shown in Fig. 1. The height is h_n = 29.2 m, where the storey and ground floor height are 3.6 m and 4.0 m, respectively. In each orthogonal direction, the building is laterally braced by two Moderate Ductile Concentrically Braced Frames (MD-CBFs) with tension-compression braces designed in compliance with the requirements of the NBC 2020 [1] and the Steel Design standard, CSA/S16:19 [11]. For MD-CBFs, the ductility R_d and overstrength reduction factors R_0 are 3.0 and 1.3, respectively. The P-Δ effect and torsional effects are considered in design.

The benchmark is the B#1 regular building (RB) shown in Fig. 1a. Changing the MD-CBFs location in relation to the center of mass (CM), the building B#2 (Fig. 1b) is torsionally-sensitive (TSB), hence irregular. Dead (D), live (L), and snow load (S), as well as cladding load are given in Fig. 1c.

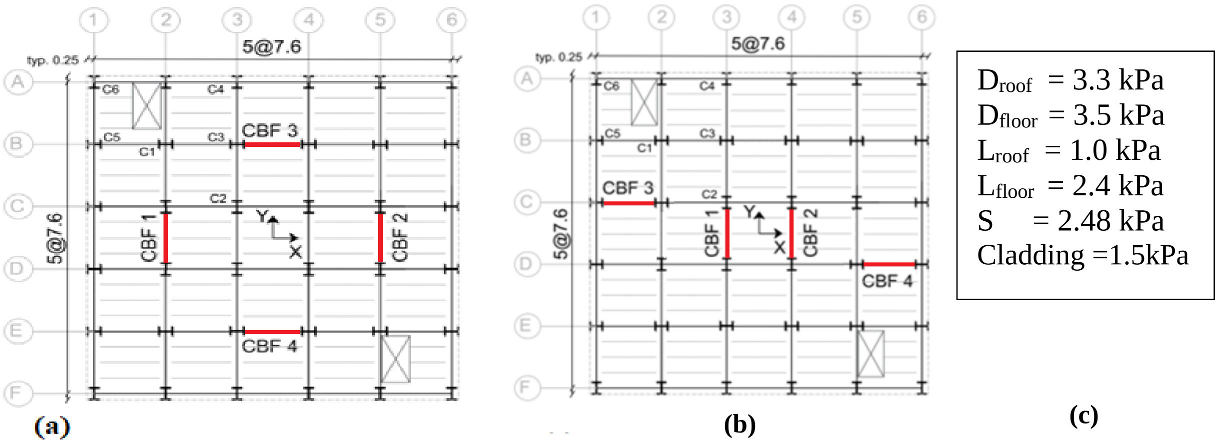


Fig. 1. Building plan and design loads: a) B #1, b) B #2, c) loads.

3.1 Design for torsion

The current building code [1] permits the application of the Equivalent Static Force Procedure (ESFP) for regular buildings under 60 m height and fundamental period less than 2.0 s in both orthogonal directions. However, for designing torsionally-sensitive buildings in seismic category SC3 and SC4, dynamic analysis is required. The SC is determined on the basis of $I_ES(0.2)$ and $I_ES(1.0)$, where $S(0.2)$ and $S(1.0)$ are the spectral ordinate at 0.2 s and 1.0 s, respectively, while I_E is the importance factor for seismic load. When $I_ES(0.2) > 0.75$ or $I_ES(1.0) > 0.3$, the building is in SC4. For Montreal site class C (stiff soil), the 2%/50 years design spectral response accelerations for 0.2, 0.5, 1.0, 2.0, and 5.0 s are 0.84, 0.49, 0.257, 0.116, and 0.0301 g. Considering the fundamental period $T_a = 0.025h_n$ it leads to 0.73 s. In design, $2T_a$ = 1.46 s is allowed when dynamic analysis is provided. The building seismic weight that includes 25% snow load is W = 48,409 kN. The base shear V_E = 2385 kN is calculated with Eq. (1), where $S(T_a)$ =0.192g, I_E = 1.0, and M_v = 1.0 is the higher mode effect on base shear.

$$V_E = I_E S(T_1) M_v W / R_d R_0 \quad (1)$$

To account for torsion effect in design, the torsional moment T_x at each level x is calculated as per Eq. (2) from [1], where F_x is the lateral seismic force at level x, e_x and $e_a = \pm 0.10D_{nx}$ are the inherent and accidental eccentricity, respectively, while D_{nx} is the building dimension perpendicular to the direction of applied lateral seismic force. It is noted that torsional-sensitivity is determined by calculating the ratio $B_x = \delta_{max}/\delta_{avg}$ for each level x and in each orthogonal direction independently. Herein, δ_{max} is the maximum storey displacement at the extreme points of the building floor at level x in the direction of applied force F_x acting at distances $\pm 10D_{nx}$ from CM at each floor, while δ_{avg} is the average of the displacements at the extreme points of the building floor at level x. In accordance with [1], the building is defined as TSB when B_x exceeds 1.7. For regular buildings ($B < 1.7$) is permitted to account on the effect of accidental torsional moments by considering a 3D numerical model subjected to dynamic analysis where the centres of mass is shifted by a distance of $e_a = \pm 0.05D_{nx}$.

$$T_x = F_x(e_x \pm 0.10D_{nx}) \quad (2)$$

To evaluate the effect of accidental torsional moments in design, the provisions presented in the ASCE/SEI 7-22 standard [2] for TSBs are also considered. Conversely to the NBC [1] requirements, in [2], the accidental eccentricity is defined as $\pm 0.05D_{nx}$. In addition, the building is classified with torsional irregularity when $B > 1.2$ and extreme torsional irregularity when $B > 1.4$. Hence, buildings with torsional irregularity shall have the torsional moment at each level amplified by a torsional amplification factor A_x given in Eq.(3), where δ_{max} and δ_{avg} were defined above.

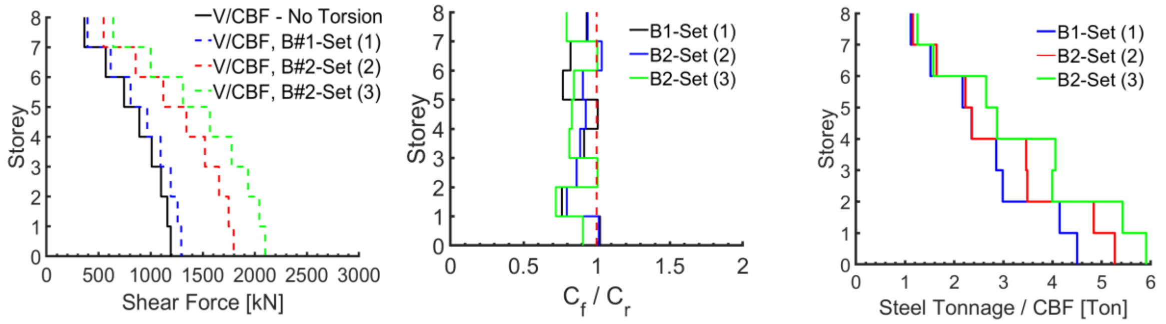
$$A_x = (\delta_{max}/1.2\delta_{avg})^2 \text{ where } 1.0 \leq A_x \leq 3.0 \quad (3)$$

In this paper, three distinct design sets are considered to evaluate the effect of torsional moments on the seismic response of RB and TSB presented in Fig. 1 such as:

- Design Set (1) with $e_a = \pm 0.05D_{nx}$ as permitted in NBC for RBs,
- Design Set (2) with $e_a = \pm 0.10D_{nx}$ as mandated in [1] for TSBs, and
- Design Set (3) with $e_a = \pm 0.05D_{nx}$ and amplification factor A_x as required in [2] for TSBs.

Members of MD-CBFs are designed in agreement with [11]. Braces of braced frame are made of hollow structural section (HSS) with F_y = 350 MPa and probable yield strength 460 MPa, while beams and columns are made of I-shape sections with yield strength F_y = 345 MPa. All HSS braces are Class 1. All columns are continuous over two stories and are pinned at the base, while all beams and braces are pin-ended. To design beams and columns of MD-CBF, the capacity design approach is applied. A 3D building model was developed using ETABS software and the ratio B was calculated at each floor. The peak value of B among floors is provided in Table 1, as well as the first three vibration mode periods resulting from eigenvalue analysis in both orthogonal directions. Analysing $T_{1\theta}/T_{1N\text{-}S}$ ratios from Table 1, a correlation with TSB is observed; hence, for $T_{1\theta}/T_{1N\text{-}S} > 2$ is more likely that the building is torsionally-sensitive, where $T_{1\theta}$ is the first mode period in torsion.

The distribution of base shear V_E along the building height with and without the consideration of shear caused by torsion is presented in Fig. 2a. As resulted, the increase of shear caused by torsion is 8% for B#1 (RB) designed with Design Set (1), 51% and 76% for B#2 (TSB) designed with Design Sets (2) and (3), respectively. For a single MD-CBF of B#2 designed with Design Set (2) and Set (3) the steel tonnage increases by 15% and 29%, respectively, when comparing with MD-CBF of regular B#1 building.



(a) **(b)** **(c)**

Fig. 2. Design characteristics of case studies following Design Sets (1-3): a) shear force, b) demand to resistance of HSS braces and c) steel tonnage per MD-CBF of B#1 and B#2.

ID	Software	*B*	$T_{1N\text{-}S}$	$T_{2N\text{-}S}$	$T_{3N\text{-}S}$	$T_{1E\text{-}W}$	$T_{2E\text{-}W}$	$T_{3E\text{-}W}$	$T_{1\theta}$	$T_{2\theta}$	$T_{3\theta}$	$T_{1\theta}/T_{1N\text{-}S}$
B#1	ETABS	1.29	1.606	0.548	0.308	1.607	0.549	0.308	1.625	0.555	0.312	1.01
Set (1)	OpenSees	-	1.492	0.498	0.281	1.524	0.509	0.287	1.525	0.510	0.288	1.02
B#2	ETABS	3.50	1.487	0.522	0.296	1.487	0.522	0.296	4.514	1.585	0.898	3.04
Set (2)	OpenSees	-	1.391	0.478	0.272	1.393	0.479	0.272	4.791	1.543	0.821	3.44
B#2	ETABS	3.50	1.352	0.467	0.261	1.352	0.467	0.261	4.103	1.417	0.792	3.03
Set (3)	OpenSees	-	1.263	0.427	0.240	1.264	0.427	0.240	4.210	1.322	0.714	3.33

Table 1: Dynamic properties of 8-storey buildings B#1 and B#2 following Design Sets (1-3).

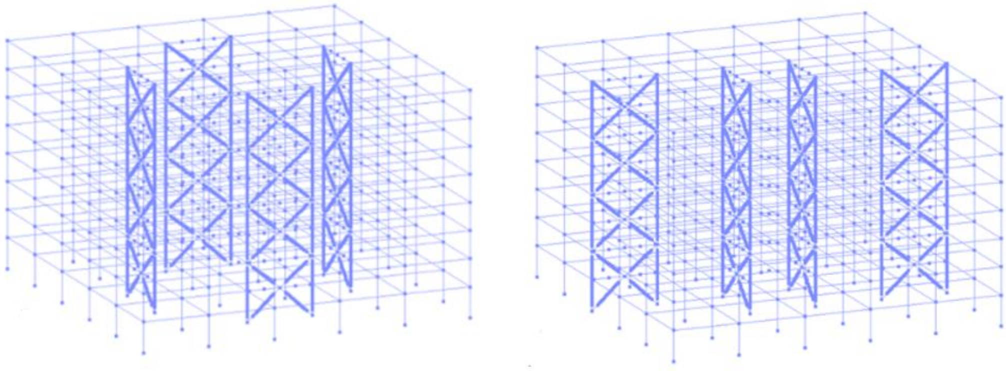
3.2 Numerical model

To assess the nonlinear behavior of case studies, a detailed 3D numerical model was developed in OpenSees [12]. Then, the 3D building model was subjected to Nonlinear Response History Analysis (NRHA). As illustrated in Fig. 3, each model includes all MD-CBFs along with girders and columns of gravity load-resisting system. Secondary beams are incorporated by applying their reactions to associated girders. The model accounts for P-δ effect, large displacements, and accidental torsion induced by shifting the mass as per the designated Design Set. Each HSS brace is modeled using 16 force-based nonlinear beam-column elements with distributed plasticity and fiber-section discretization, and three integration points per element. An initial imperfection of 1/500 of brace length was assigned to account for out-of-plane buckling of brace. Moreover, HSS braces are discretized into 240 quadrilateral patches, with 40 fibers per edge segment and 20 fibers for rounded corners. Material nonlinearity is modelled using *Steel02* material with isotropic strain hardening (1%). A fatigue material with a fatigue ductility exponent of m =-0.5 and a ductility coefficient ε_0, calibrated based on equation given in [13] is wrapped to the parental *Steel02* material to capture the low-cycle fatigue failure. The I-shape columns and beams of MD-CBFs are simulated using eight and four nonlinear beam-column elements with distributed plasticity, respectively. Columns are assigned an initial imperfection of 1/1000 of their length to account for in-plane and out-of-plane buckling. Each I-shaped column and beam section is discretized into 120 fibers. Columns extend continuously over two stories as per [11].

Pin connections are modeled as *two-node Zero-Length* elements with highly stiff elastic material in translational and torsional directions. Free rotation about in-plane and out-of-plane axes is modelled using an elastic material with negligible tangent modulus. The brace-to-frame gusset plate connection is modeled with two rotational springs (linear in-plane and nonlinear out-of-plane) and one torsional spring within a *Zero-Length* element. Floors are modeled as rigid diaphragms, with the center of mass (CM) at each floor acting as the master node. Translational masses $M_{x,i}$ and $M_{y,i}$ are lumped at the CM of floor *i*, while the mass moment of inertia, $I_{CM,i}$, is calculated as:

$$I_{CM,i} = (W_i/g)(L_x^2 + L_y^2)/12 \tag{4}$$

where L_x and L_y are the floor dimensions, and W_i is the seismic weight of that floor. Rayleigh damping is applied with a 2% damping ratio for the first and third translational modes, and is assigned only to elastic members. The analysis is performed using the Newton algorithm with line search.



(a) (b)

Fig. 3. Three-dimensional OpenSees numerical models for the 8-storey: a) B#1 building, b) B#2 building.

3.3 Seismic data

Due to lack of historical ground motions (GM_{H}s) in Eastern Canada, seven synthetic GMs were selected from database www.seismotoolbox.ca. These GMs simulate magnitude 7 earthquakes at epicentral distances from 13.8 to 50.3 km for Site Class C in Montreal. Each record lasts 18.0 s and additional 12.0 s of zero padding were added to account on free vibrations. The scaling of GMs is accomplished such that the mean response spectrum remains above 90% of the 2%/50 years design spectrum over the period of interest $0.15T_1 – 2.0T_1$. For TSB, the torsional mode period, $T_{1\theta}$, exceeds the translational mode period $T_{1,N\text{-}S}$; hence, the later period was considered in the scaling procedure. Figure 4 shows the design spectrum alongside the response spectra of scaled GM suite and their mean.

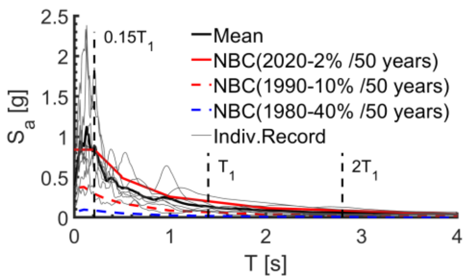
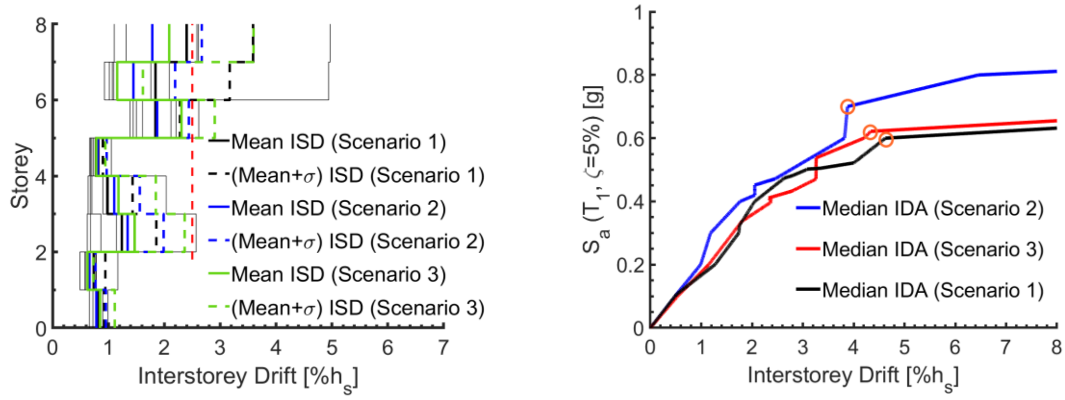


Fig. 4. Design spectrum for Montreal, Site Class C, and the response spectra of scaled GM suite.

The selected synthetic GMs have only one component. To investigate the building response to biaxial excitation, a second horizontal component is required. To derive the second GM component from the primary synthetic GM, a study implying historical GM_{H}s is conducted. The regular building B#1 was re-designed for Vancouver, B.C. and GM_{H}s considered are provided in Table 2. A 3D building model was subjected to NRHA under three Biaxial Excitation Scenarios: (1) historical records with both horizontal components, (2) the main component of a historical GM_H and 30% of the same component in the orthogonal direction, and (3) the main component of a historical GM_H and 80% of the same component in the orthogonal direction. It is worth noting that the same scaling factor was applied for both horizontal components of a GM_H. Herein, Scenario (2) was selected to emphasize on the feasibility of "30%" rule provided in NBC [1]. The seismic response of B#1expressed in terms of interstorey drifts (ISDs) resulted under scaled GM_Hs at design level (D.L.) as per Scenarios (1-3) is plotted in Fig. 5a. It is concluded that the peak of mean ISD resulted under real biaxial excitation (Scenario 1) is similar to that obtained under biaxial excitation as per Scenario (3). Furthermore, B#1 building was subjected to Incremental Dynamic Analysis (IDA) as per the methodology presented in [14]. The median IDA curves resulted under biaxial excitation as per Scenarios (1-3) are plotted in Fig. 5b. As expected, the most critical is the response of B#1 under Scenario (1) followed by Scenarios (3) and (2), respectively. From this analysis is concluded that the "30%" rule can be applied in the elastic range but is not feasible in highly nonlinear range. Because the median IDAs resulted under Scenarios (3) and (1) are similar, Scenario (3) is considered appropriate for further analyses. Therefore, for buildings in Montreal, the pair horizontal component of scaled synthetic GMs was set to 80% of the main component.

Event	M_W	Station	R (km)	Comp (°)		PGA (cm/s^2)		S.F.
Oct. 18, 1989, Loma Prieta	6.9	APEEL 9 - Crystal Springs	41	137	227	115.5	103.5	2.6
Oct. 18, 1989, Loma Prieta	6.9	Anderson dam (downstrem)	20	243	333	245.4	239.7	2.5
Oct. 18, 1989, Loma Prieta	6.9	Gilroy Array #3	13	0	90	531.7	362	1.3
Oct. 18, 1989, Loma Prieta	6.9	Palo Alto - SLAC Lab	31	212	302	378.2	341.5	1.35
Jan. 17, 1994, Northridge	6.7	Castaic - Old Ridge Route	21	90	360	557.1	504.2	1.0
Jan. 17, 1994, Northridge	6.7	LA - UCLA Grounds	14	360	90	464.6	272.4	1.9
Jan. 17, 1994, Northridge	6.7	Moorpark - Fire Sta	17	180	90	286.2	189.3	2.1

Table 2: Selected ground motions for Vancouver, B.C., Site Class C.



(a) (b)

Fig. 5. Response of B#1 under 3 biaxial excitation scenarios: a) ISD at D.L., b) median IDA curve per scenario.

4 NONLINEAR SIMULATION AT DESIGN LEVEL AND BEYOND

Both B#1 (RB) and B#2 (TSB) are subjected to NRHA using OpenSees. To capture the effect of biaxial excitation, different mass eccentricity configurations are considered on the 3D model. To analyse the seismic response of these buildings the selected scenarios are:

- (a) no accidental eccentricity (the mass is lumped at CM with $e_{ax} = 0$) and the response is obtained under the uniaxial GMs (N-S direction) noted 100% GM_{N-S};
- (b) mass is lumped at CM shifted by $e_{ax} = \pm 0.05D_{nx}$ for buildings B#1 and B#2 (Design Set 3) and $e_{ax} = \pm 0.10D_{nx}$ for B#2 (Design Set 2), while the response is obtained under the uniaxial GMs (N-S direction) noted 100% GM_{N-S}; and
- (c) mass is lumped at CM shifted by $e_{ax} = \pm 0.05D_{nx}$ for buildings B#1 and B#2 (Design Set 3) and $e_{ax} = \pm 0.10D_{nx}$ for B#2 (Design Set 2), while the response is obtained under biaxial ground motions 100% GM_{N-S} and 80% GM_{E-W}.

It is worth mentioning that the available synthetic GMs for Montreal have only a single horizontal component and the second is derived from the main component. The nonlinear seismic responses of B#1 and B#2 buildings under GMs scaled at design level (D.L.) that are associated with Scenarios (a-c) are shown in terms of ISDs in Fig. 6.

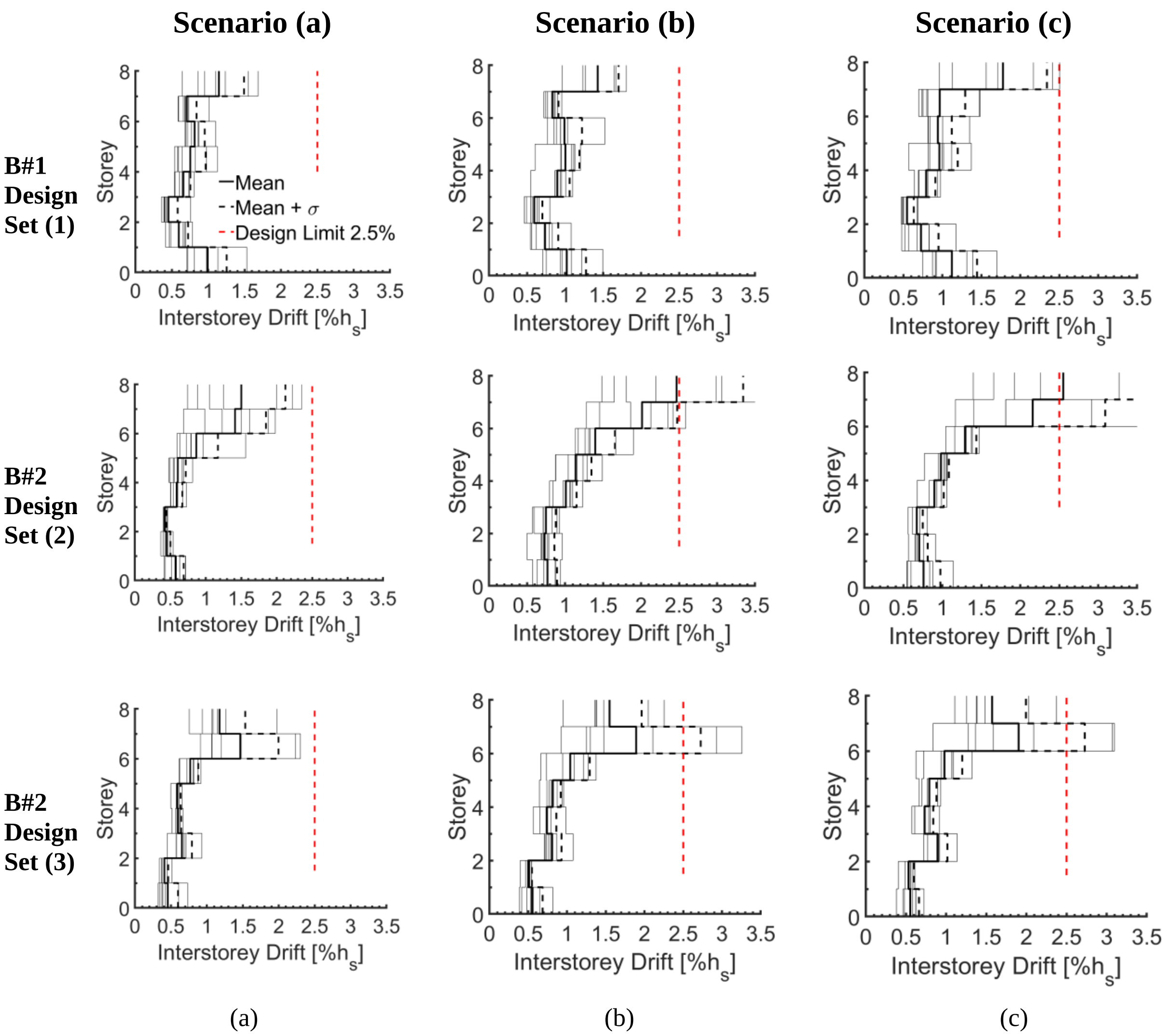


Fig. 6. Seismic response of 8-storey RB (B#1) and TSB (B#2) buildings plotted in terms of ISD under GM suite scaled at D.L., considering: a) $e_{a,x}$=0% & 100% GM_{N-S}; b)- c) $e_{a,x}$=5% for B#1 and B#2 (Design Set 3) and $e_{a,x}$=10% for B#2 (Design Set 2) under 100% GM_{N-S}; and 100% GM_{N-S} & 80% GM_{E-W}, respectively.

Following Scenario (a), both B#1 and B#2 with Design Set (2) show a similar response; hence, the peak of mean ISD is 1.2%h_s and 1.5%h_s, respectively, and occurs at top floor. Conversely, the B#2 building designed as per Design Set (3) shows the peak of mean ISD (1.45%h_s) at 7th floor. Both the peak of mean and the peak of mean plus one standard deviation M+σ ISDs are within the 2.5%h_s code limit. Thus, for e_{ax} = 0, the peak of mean ISD of RB and TSB are similar.

According with Scenario (b), the B#1 and B#2 designed with Design Set (2), show the peak of mean ISD of 1.42%h_s and 2.47%h_s, respectively, at top floor. Conversely, the B#2 with Design Set (3) shows lower peak of mean ISD (1.89%h_s) triggered at 7th floor.

In Scenario (c), the peak of mean ISD of B#1 and B#2 with Design Set (2) occurred at top floor and increased from 1.78%h_s for B#1 (RB) to 2.55%h_s for B#2. However, for B#2 with Design Set (3), the peak of mean ISD decreased to 1.9%h_s and occurred at 7th floor. From Fig. 6 it can be concluded that the TSB exhibited larger ISDs than the RB. In addition, B#2 designed with Design Set (3) shows lower ISD when compared with the response of B#2 designed with Design Set (2).

To assess the collapse capacity of 8-storey B#1 and B#2 buildings, Incremental Dynamic Analysis (IDA) is applied as per the methodology provided in [14]. Although the IDA curves for the 8-storey building are not provided herein, from previous analyses on 4-storey building with the same floor plan resulted that the TSB designed as per Design Set (2) failed to pass the collapse safety criteria under biaxial excitation [15]. Moreover, the TSB designed in accordance with the ASCE-based Design Set (3) is able to mitigate the adverse effects of biaxial excitation more effectively and the building is able to pass the collapse safety criteria as required in the methodology provided in FEMA P-695 [16].

5. STRUCTURAL ASSESSMENT OF REGULAR BUILDING

In Canada, the largest building stock was built in the 1980s and the structural design followed the 1980 edition of the National Building Code of Canada [9]. At that time, Montreal was in seismic zone 2, where the associated A parameter is the ratio of peak horizontal ground accelerations to acceleration due to gravity. The acceleration amplitude corresponds to 0.01 probability of exceedance per annum (40%/50 years). The minimum design base shear, V_{80}, is:

$$V_{80} = A \ x\ S\ x\ K\ x\ I\ x\ F\ x\ W \tag{5}$$

where A is the design ground acceleration given as 0.04 for Zone 2 (Montreal), S is the seismic response factor equal to min(0.5/$\sqrt{T_a}$, 1.0) and $T_a = 0.09h_n/\sqrt{D}$ where D is the building dimension in the direction parallel to the applied seismic forces, K is a coefficient that reflects the type of construction, damping and ductility (K=1 for tension-compression braces), I and F are the importance and foundation factors, respectively, while W is the seismic weight that includes the 25% of roof snow load (SL).

For the case study presented in Fig. 1a, T_a = 0.42 s with D =38.5 m and h_n = 29.2 m that leads to S = 0.77. Applying Eq. (5) with I = 1.0 and F = 1.0 (stiff soil), it results V_{80} = 1490 kN, where W = 48,350 kN is slightly lower because of snow load equal to 2.32 kPa. Moreover, according to [9], there are three load combinations: (1) 1.25DL+1.5LL, (2) 1.25DL+ 1.5E, and (3) 1.25DL+ 1.05LL+1.05E. From load combination (2), the factored base shear $V_{f,80}$ is 2235 kN. However, $V_{f,80}$ represents near elastic demand and no ductility nor capacity design approach is further accounted for in the CBF design. It is noted that the base shear $V_{E,20}$ = 2385 kN calculated with Eq. (1) considering R_dR_0 =3.9 is similar to $V_{f,80}$. However, $V_{f,80}$ represents only 31% of $V_{E\ elastic,20}$ = 7155 kN associated with the current code [1]. Moreover, the expression of accidental eccentricity, e_a =±0.05D_{nx} is the same as that given in the current building code for regular buildings. In the calculation of P-Δ, in [1], the interstorey drift is multiplied by R_d leading to a larger P-Δ effect.

In the 1980s, steel beams and columns were made of W-shapes using the CSA-G40.21 300W steel with yield strength F_y = 300 MPa and ultimate strength F_u = 450 MPa, which show lower yield strengths than those fabricated after the 1990s. Moreover, braces were made of back-to-back angles with F_y = 300 MPa or hollow structural sections (HSS) with steel yield strength F_y = 345 MPa and ultimate strength F_u = 448 MPa. The yield strength of gusset plates was 300 MPa. In this case study, columns of the 8-storey braced bay are pinned connected at each two floors and all beams of CBFs were pinned connected to column flanges, while all HSS braces were pinned at both ends.

The seismic design of an existing 8-storey office building was conducted using the ESFP according to building code [9] and CSAS16.1-M78 steel design standard [10], where the limit states design philosophy was first introduced. Accordingly, a concentrated load $F_{t,80}$ = $0.004V_{f,80}(h_n/D_s)^2$ is applied at the roof level when $h_n/D_s > 3$, but $F_{t,80}$ needs not exceed $0.15V_{f,80}$ where D_s is the dimension of the lateral force resisting system. The remaining seismic load, ($V_{f,80}$-$F_{t,80}$), is distributed along the building height as a function of the relative product of the seismic weight and the elevation from the ground at the level under consideration. In this case study, h_n/D_s =3.84 and $F_{t,80}$ =132 kN. A comparison of brace compression resistance resulted from the current design ($C_{r,20}$) versus the old design ($C_{r,80}$) is presented in Fig. 7. As depicted, the old HSS braces are under design by rations between 18% and 65%. Although not presented herein, these ratios increase substantially for the brace-to-frame connections as reported in [8, 17]. Figure 7 also presents the demand-to-capacity ratio ($C_{r,20}/C_{r,80}$) for CBF columns which is in the range 2.52 - 2.82. It is noted that in NBC 2020, the capacity of column members is examined for (i) cross-sectional strength, (ii) overall member strength and (iii) lateral torsional buckling strength. Whiten the interaction equation, the term C_f/C_r has the larger weight while the term $0.85U_1M_f/M_r = 0.18U_1$ (where U_1 ~ 1.0 and $M_f = 0.2M_p$) has the lesser contribution. Meanwhile, applying the capacity design approach, the axial force triggered in column results from brace members (C_{prob} and T_{prob}) projection, where C_{prob} and $T_{prob} = A_gR_yF_y$ are the probable buckling compression strength and the probable tensile strength, respectively, were calculated with R_yF_y = 460 MPa and A_g is the area of brace. However, the axial force in the column cannot be greater than that transferred from braces considering $V_{E\ elastic}$. In the 1980s, an additional bending moment in the direction of the braced bay of $0.2M_p$ was not required. The same interaction equation is considered for the beam design. It is noted that at floors 1, 3, 5, and 7, braces intersect the beam at its mid-span length. Applying the capacity design required by the current design standard, the governing scenario is reached when all HSS compression and tensile braces carry the probable post-buckling compression strength $C'_{prob} = 0.2T_{prob}$ and the probable tensile strength T_{prob}, respectively. In this scenario, braces are not provided support to the beam mid-span. Projecting all member forces with respect to a vertical axis, an unbalanced vertical load acts at braces-to-beam intersection and generates a large bending moment on beams in addition to the bending moment from gravity loads. Axial tensile and compression forces are also transferred to these beams.

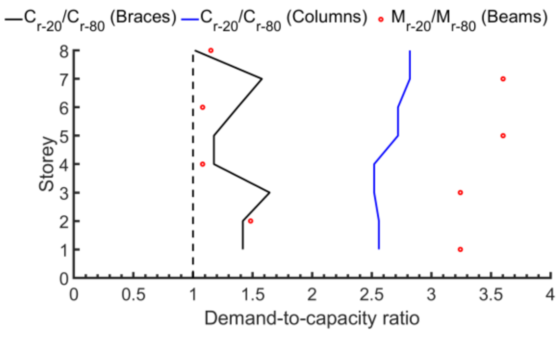


Fig. 7. Demand to capacity ratio of braces, columns, and beams of CBF.

Conversely to column design, the term $0.85U_1M_f/M_r$ has the larger weight in the interaction equation used for beam design verification. For beams, the demand to capacity ratio expressed as $M_{r,20}/M_{r,80}$ is between 3.2 and 3.5. However, the beams at 2, 4, 6, and 8 floors are under design by only 5%-10%.

6 CONCLUSIONS

This paper presents the behaviour of a torsionally-sensitive 8-storey braced frame building against a counterpart regular building. A comprehensive 3D building model was developed in OpenSees and subjected to uniaxial and biaxial excitation. Seismic assessment of the 8-storey RB designed as per the 1980s code and standard is also presented. The findings are:

- The increase of shear caused by torsion is 8% for RB designed with 5% accidental eccentricity, 51% for TSB designed with NBC-based Design Set (2) and 76% for TSB designed with the ASCE-based Design Set (3). For a single MD-CBF of TSB designed with Design Sets (2) and (3) the steel tonnage increases by 15% and 29%, respectively, when comparing with that of MD-CBF of RB.
- Analysing the RB under historical GMs it was found that under biaxial excitation with real GM horizontal components, the collapse capacity is about 20% lower than in the case when biaxial excitation includes the “30%” rule (100% GM in one direction and 30% GM in the orthogonal direction). Thus, the “30%” rule is not feasible in highly nonlinear range.
- When torsion caused by accidental eccentricity was not considered in design and the building was subjected to uniaxial excitation, the peak of mean ISD of RB is similar with that of TSB. Conversely, the response of TSB designed with ASCE-based Design Set (3) versus Design Set (2) shows improved torsional resistance under biaxial excitation and 6% increase of peak of mean ISD when comparing with RB. However, the peak of mean ISD of TBS designed with Design Set (2) shows 43% increase when comparing with RB.
- From previous study conducted on a 4-storey building with the same floor plan, the collapse margin ratio (*CMR*) of RB under uniaxial and biaxial excitation is almost the same. Conversely, for TSB designed as per Design Set (2) and the ASCE-based Design Set (3) the *CMR* dropped substantially under biaxial versus uniaxial excitation, emphasizing the vulnerability of TSBs to bidirectional loading. In addition, the TSB with Design Set (2) does not pass the collapse safety criteria.
- From seismic assessment of regular building B#1 designed according with the NBC 1980 and the associated steel design standard it results that HSS braces and columns of older building yield demand to capacity ratios in the range of 1.1-1.57 and 2.52-2.82, respectively. In addition, beams of CBF that are supported by braces at their mid-span (e.g. floors 1, 3, 5, and 7) have the demand to capacity ratios between 3.25 and 3.6, while the remaining beams are under design by only 5-10%.

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