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Torsional effects in symmetrical steel buckling restrained braced frames: evaluation of seismic design provisions

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Abstract. The effects of accidental eccentricity on the seismic response of four-storey steel buildings laterally stabilized by buckling restrained braced frames are studied. The structures have a square, symmetrical footprint, without inherent eccentricity between the center of lateral resistance (CR) and the center of mass (CM). The position of the bracing bents in the buildings was varied to obtain three different levels of torsional sensitivity: low, intermediate and high. The structures were designed in accordance with the seismic design provisions of the 2010 National Building Code of Canada (NBCC). Three different analysis methods were used to account for accidental eccentricity in design: (1) Equivalent Static Procedure with static in-plane torsional moments assuming a mass eccentricity of 10% of the building dimension (ESP); (2) Response Spectrum Analysis with static torsional moments based on 10% of the building dimension (RSA-10); and (3) Response Spectrum Analysis with the CM being displaced by 5% of the building dimension (RSA-5). Time history analyses were performed under a set of eleven two-component historical records. The analyses showed that the ESP and RSA-10 methods can give appropriate results for all three levels of torsional sensitivity. When using the RSA-5 method, adequate performance was also achieved for the low and intermediate torsional sensitivity cases, but the method led to excessive displacements (5-10% storey drifts), near collapse state, for the highly torsionally sensitive structures. These results support the current provisions of NBCC 2010.

Keywords: earthquakes; symmetry; buckling restrained braced frames; accidental eccentricity; torsional sensitivity.

1. Introduction

Building codes recognize the importance of considering seismically induced torsional moments in structures in design so that their adverse effects on seismic performance can be minimized. Torsional behaviour is due to the inherent eccentricity between the center of mass (CM) and the center of lateral resistance (CR), as well as other effects including the rotational components of ground motions and the uncertainty in the distribution of mass, stiffness and yield strength of structural components (Chopra and De la Llera 1994). In codes such as the National Building Code of Canada (NBCC) (NRCC 2010), ASCE 7 (ASCE 2010), Eurocode 8 (EC8) (CEN 2004) or NCh433 (INN 2009), these additional effects are taken into account in design by means of the

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accidental design eccentricity (ADE). These codes have different procedures to introduce the ADE. They also have different definitions of torsionally regular (non-sensitive) or irregular (sensitive) buildings.

In the last decades, seismic torsional behaviour of building structures has been studied by many researchers. Most studies involved simplified one-storey asymmetric building structures (Anagnostopoulos *et al*. 2010) and their conclusions are difficult to extrapolate to 3D multi-storey buildings with seismic-force-resisting system (SFRS) exhibiting specific stiffness, strength and hysteretic characteristics such as steel concentrically braced frames (CBFs), eccentrically braced frames (EBFs) or buckling restrained braced frames (BRBFs). Torsion analysis and design of multi-storey buildings is thus currently an attracting field of research because of the limited understanding of the nonlinear torsional behaviour of different SFRSs (Basu *et al*. 2014, DeBock *et al*. 2013, Kyrkos and Anagnostopoulos 2013, 2011a, b, De-la-Colina *et al*. 2011). For steel braced frames, Erduran and Ryan (2011) noted that due to the ADE, yielding takes place in one bracing bay, leading to a "flexible" building side with a dynamic shift of the CR towards the "stiff" side, thereby increasing the eccentricity between the CM and the CR. This phenomenon is not observed in linear analyses typically used in conjunction with ductility force reduction factors in design practice. Kyrkos and Anagnostopoulos (2011a) observed that the ductility demand on the braced frames located on the flexible side is much more important than that on the bracings on the stiff side. For asymmetric buildings, the flexible side is the one where the bracing elements are the farthest away from the CR. For these structures, design recommendations have been suggested to increase the strength of the vertical bracing bents on the flexible side and decrease that of those located on the stiff side to achieve a more uniform ductility demand (Kyrkos and Anagnostopoulos 2011b). For symmetric buildings, a flexible side cannot be readily identified. It corresponds to the side where translational motion due to floor rotation is additive to the translational response. Stathopoulos and Anagnostopoulos (2010) noted that the ADE has more influence on the response of symmetric buildings than buildings with inherent torsion, regardless of the magnitude of the ADE.

This study contributes to the understanding of the seismic torsional behaviour of symmetrical steel buildings due to accidental eccentricity. The structures studied are four-storey buildings with BRBFs as the SFRS in both directions. Thus, the study is limited to structures for which the seismic response is mainly governed by their first vibration modes. The structures were configured to exhibit three different levels of torsional sensitivity: non sensitive (regular), at the limit between non sensitive and sensitive, and sensitive (irregular) structures. All structures were designed in accordance with the provisions of NBCC 2010 and three-dimensional dynamic nonlinear time histories analyses were performed including the nonlinear hysteretic behaviour of the bracing members. Overall, this research pursues two main objectives: (1) evaluation of the influence of torsional sensitivity on the response of multi-storey steel buildings with representative geometry and SFRS subjected to strong ground motions; and (2) evaluation and comparison of the three different analysis methods that are specified in NBCC 2010 to account for accidental torsional effects. These methods are: (1) the Equivalent Static Procedure with static in-plane torsional moments assuming a mass eccentricity of *10%* of the building dimension (ESP); (2) the Response Spectrum Analysis with static torsional moments based on *10%* of the building dimension (RSA-10); and (3) the Response Spectrum Analysis with the CM being displaced by *5%* of the building dimension (RSA-5). No such comparative study using three-dimensional nonlinear dynamic analysis has been realized to evaluate the adequacy of these three approaches. In addition, the research aims at verifying if the structure displacements, as predicted at the design phase on the basis of the equal displacement principle adopted in the NBCC, correspond to those obtained from nonlinear dynamic time histories analyses. An overview of the provisions included in that and other modern codes for torsional effects in seismic analysis and design is first presented. The design and modelling of the prototype structures are then described. The results of the analyses are presented, with focus on the computed peak storey drifts. The main findings of the study are summarized in the conclusions.

2. Code seismic provisions for in-plane torsion

A key parameter to establish the torsional sensitivity of buildings is the ratio between the uncoupled torsional frequency, ω_θ, to the uncoupled translational frequency, ω_y, that is $\Omega_R=\omega_\theta/\omega_y$ (Humar *et al*. 2003). The factor Ω_R also reflects the ratio of the building torsional stiffness to its translational stiffness. Buildings with $\Omega_R \geq 1$ are considered as “rigid”, regular or insensitive to torsion. Buildings with $\Omega_R < 1$ are considered as “flexible”, irregular or sensitive to torsion. For buildings with rigid floor diaphragms, Humar *et al*. (2003) proposed a relationship between Ω_R and a new parameter B that can be easily computed at the design stage. That parameter is the maximum of all values of B_x computed at every level x of the structure

$$B_x = \frac{\delta_{\max}}{\delta_{avg}} \tag{1}$$

where $\delta_{\max}$ is the maximum storey displacement at the extreme points of the structure at level x, in the direction of the earthquake, as induced by the equivalent static forces acting at a distance ±0.10 D_n from the center of mass, δ_{avg} is the average of the displacements of the extreme points of the structure at level x produced by the above forces, and D_n is the building dimension at level x perpendicular to the direction of the earthquake. B is computed for both perpendicular directions and the maximum value is retained to classify the building as regular or irregular in torsion: a building is considered torsionally non sensitive (regular) if $B<1.7$ and torsionally sensitive (irregular) when $B>1.7$. Due to ADE, a perfectly symmetrical building may therefore be irregular in torsion if it has relatively low torsional stiffness, as it is the case for core type SFRSs with bracing elements located close to the geometric center.

The B parameter has been adopted in the NBCC and torsional irregularity influences the design of structures located in moderate and high seismic zones, including the choice of the analysis method and the in-plane torsional moments due to ADE. For regular buildings, the equivalent static procedure (ESP) is allowed for structures that are up to 60 m in height and have a fundamental period not exceeding 2.0 s. In this case, the accidental torsional moment at level x is defined as (±0.10 D_n) F_x, where F_x is the lateral seismic load at that storey. Alternatively, a 3D response spectrum analysis may be performed introducing the ADE by shifting the CM by ±0.05 D_n. For torsionally sensitive structures, response spectrum analysis is required instead of the ESP and ADE is accounted for by means of accidental torsional moments defined as (±0.10 D_n) F_x, where F_x are the seismic loads determined from dynamic analysis or ESP. In all the above methods, the same sign can be taken over the building height for the eccentricity.

Table 1 compares the minimum accidental torsional moments specified in the NBCC and three other modern codes. In ASCE 7-10, a smaller eccentricity is specified (0.05 D_n) but the ADE needs to be multiplied by an amplification factor, A_x, in regions of high seismicity and under

Table 1 Comparison of in-plane torsional moments due to accidental design eccentricity (ADE)

Analysis Method	Building Codes			
	Canada	United States	Europe	Chile
	NBCC 2010	ASCE 7-10	EC8-2004	NCh433-2009
ESP	$\pm 0.10\ D_n$	$A_x\ (\pm 0.05\ D_n)^1$	$\pm 0.05\ D_n$	$\pm 0.10\ D_n\ (Z/h)$
RSA	$\pm 0.10\ D_n$ or CM shift of $0.05\ D_n$	$A_x\ (\pm 0.05\ D_n)^1$ or CM shift of $0.05\ D_n$	$\pm 0.05\ D_n$	$\pm 0.10\ D_n\ (Z/h)$ or CM shift of $0.05\ D_n$

[1]For torsional irregular structures assigned to Seismic Design Category *C*, *D*, *E* or *F*

$$A_x = \left(\frac{\delta_{\max}}{1.2\delta_{avg}} \right)^2$$, and $1.0 \leq A_x \leq 3.0$; otherwise $A_x = 1.0$

conditions of torsional irregularity. The purpose of A_x is to counter the possible dynamic shift of the CR due to unequal yielding of the SFRS components creating greater torsional rotations. Previous studies have shown that this amplification factor may not be effective for controlling the building response while increasing the amount of materials in the structures (Stathopoulos and Anagnostopoulos 2010). Eurocode 8 provisions are similar to those in ASCE 7 except that ADE is not amplified in the ESP. If the RSA method is used, ADE is considered only through static in-plane torsional moments. In the NCh433 Chilean building code, ADE provisions are same as in the NBCC except that the static torsional moments are proportional to the elevation of the storey in the building, Z, with respect to the building height, h. The Z/h ratio is introduced to account for the reduced likelihood that accidental eccentricities be in the same direction at every level in multi-storey buildings (De-la-Colina *et al*. 2011). This reduction factor also takes into consideration the fact that the seismic forces induced by the higher modes of vibration are not all acting in the same direction over the structure's height, as it is the case for the fundamental vibration mode.

In the NBCC, torsional sensitivity may also have an impact on the structure's seismic lateral resistance when using the dynamic response spectrum analysis method. In that method, the results of the analysis must be scaled such that the base shear from RSA is at least equal to 0.8 times the base shear from ESP. For torsional sensitive structures (B>1.7), scaling must be performed with respect to 100% of the static base shear force.

3. Buildings

3.1 Buildings description

Three four-storey symmetrical office buildings with a square floor plan are examined in this parametric study. As shown in Fig. 1, all three buildings have a regular steel framing and lateral resistance and stability along each orthogonal direction are provided by two steel braced frames.

All three structures are identical except for the in-plan position of the braced frames which is varied to create three cases of torsional sensitivity, as discussed in the next paragraph. Buckling restrained braced frames (BRBFs) were selected for this study. Due to the high ductility of the bracing members, the system can be designed for relatively low lateral resistance. Moreover, the system typically exhibits limited overstrength. These two conditions makes the system prone to

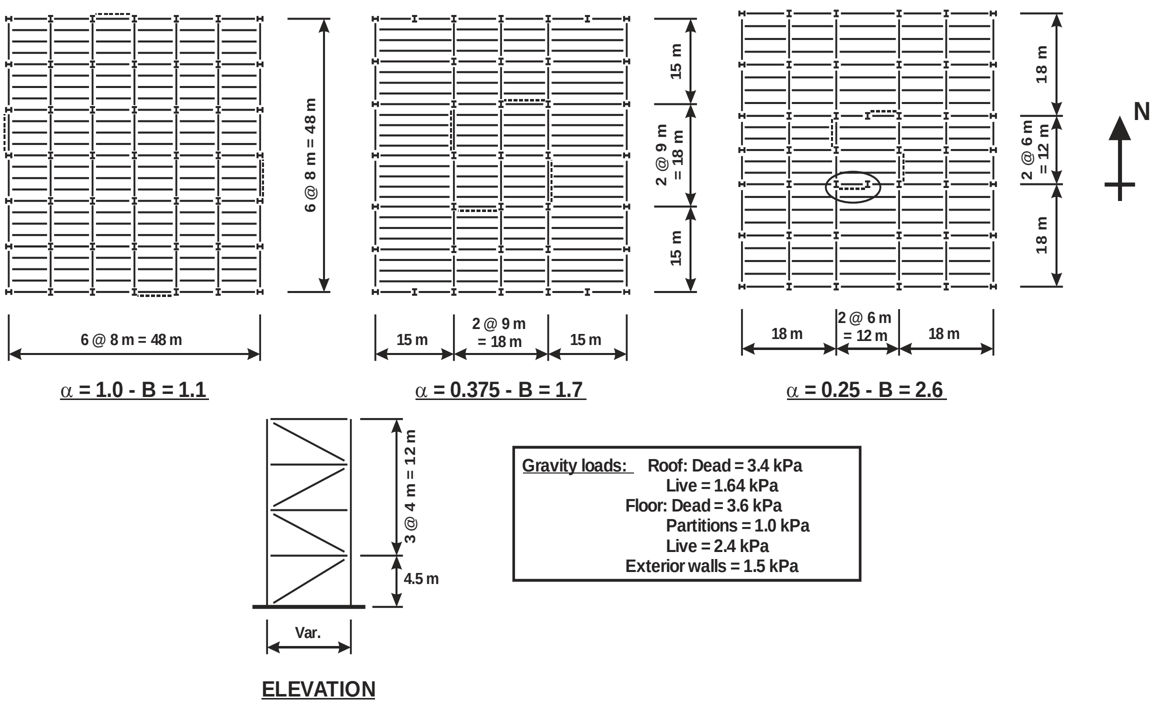

Fig. 1 Geometry and design gravity loads of the buildings studied (the center of mass and the center of lateral resistance are located at the geometric center of the buildings)

large inelastic excursions in case of excessive demand resulting from inadequate vertical or horizontal distribution of the lateral resistance. The system is therefore a good candidate to verify the appropriateness of design provisions for torsion. As shown in Fig. 1, a single diagonal bracing configuration was adopted for all frames. The design gravity loads are shown in the figure. The roof and floors are built with 76 mm deep steel deck with a 63 mm thick concrete slab topping. Rigid floor diaphragm conditions therefore exist at every level.

For symmetrical structures as studied herein, the *B* factor reflects the ratio between the rotational and the translational stiffness of the structure. Because of the symmetry, all four braced frames in the structures are designed for the same lateral loads, regardless of the seismic provisions or analysis method used in design. Hence, all braced frames in the structures studied have the same lateral stiffness and the CR is located at the center of the building. In each direction, the *B* factor only varies as a function of the spacing between the two braced frames resisting loads in that direction. In Fig. 1, the parameter α is equal to the braced frame spacing relative to the building width D_n. The three braced frame arrangements shown in the figure were selected to reflect the following three torsion sensitivity cases: (1) B=1.1, which represents a torsionally stiff building, as obtained by placing the bracing bents on the perimeter walls (α=1.0); (2) B=1.7, which represents the limit between torsionally sensitive and non-sensitive structures in the NBCC, as obtained by placing the braced frames at an intermediate position (α=0.375); and (3) B=2.6, which represents a torsionally sensitive building, as obtained by placing the braces near the geometric center of the structure (α=0.25).

3.2 Building design

The design of all buildings is performed according to the NBCC 2010 requirements. In the NBCC 2010, the minimum design lateral earthquake force, V, is given by

$$V = \frac{S(T_a)M_V I_E W}{R_d R_o} \tag{2}$$

where S is the design spectrum, T_a is the period to be used in design, M_V is a factor that accounts for the higher modes effects on the base shear, I_E is the importance factor, R_d and R_o are respectively the force modification factors related to ductility and overstrength. The structures were assumed to be located on a class *C* (firm ground) site in Vancouver, British Columbia. The design spectrum for this site is given in Fig. 2. For steel braced frames, the period T_a is taken equal to the value given by the empirical formula: T_{emp}=0.025 h_n, where h_n is the building height. Alternatively, one can use the period obtained from a dynamic analysis in the first translational mode along the direction analyzed, except that T_a cannot exceed 2 T_{emp}. For the structures studied herein, h_n=16.5 m, which gives T_{emp}=0.41 s and 2 T_{emp}=0.83 s. In the design process, dynamic analyses were performed to obtain the buildings periods. The computed periods along the N-S direction for the final designs are given in Table 2. Because of the building symmetry, the braced frames in both orthogonal directions are nearly identical and only the properties along the N-S direction are presented in the table. For all structures, the first translational mode periods $T_{1,\ N\text{-}S}$ exceed the upper limit of 2 T_{emp} and S=0.43 corresponding to a period of 2 T_{emp}=0.83 s was therefore used. Due to this code period limitation, structures with longer periods possess additional lateral strength compared to those having shorter periods. In Eq. (2), M_V=1.0 for T_a less than 1.0 s. In this study, the buildings were assumed to be of the normal importance category and the importance factor I_E=1.0. In the NBCC, the seismic weight, W, corresponds to the dead load plus 25% of the roof snow load. For all structures, it is equal to 41220 kN. For buckling restrained braced frames, the R_d and R_o factors are respectively equal to 4.0 and 1.2. Hence, V=0.091W for all buildings.

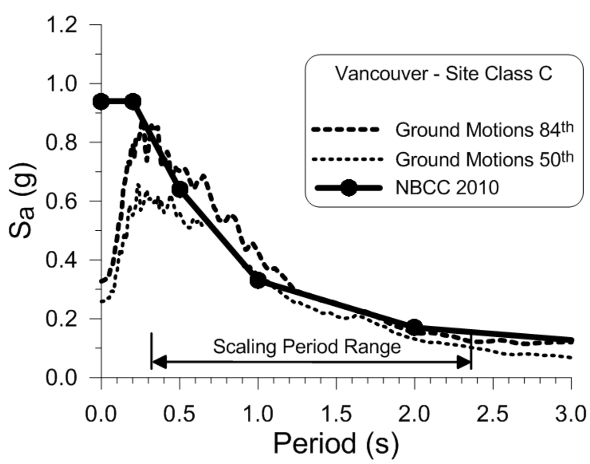

Fig. 2 Design spectrum and statistics of the 5% damped geometrical mean spectra of the scaled pairs of records

Table 2 Properties of the prototype building structures

B	$T_{1,N\text{-}S}$ (s)	$T_{2,N\text{-}S}$ (s)	$T_{1\theta}$ (s)	$T_{2\theta}$ (s)	Ω_R	$T_{1\theta}$ / $T_{1,N\text{-}S}$	$T_{2\theta}$ / $T_{2,N\text{-}S}$	V_y/W^a	V_y/V^b
					ESP				
1.1	1.05	0.39	0.63	0.24	1.66	0.60	0.61	0.113	1.240
1.7	0.96	0.36	1.57	0.59	0.62	1.64	1.62	0.129	1.417
2.6	0.92	0.33	2.25	0.75	0.42	2.44	2.31	0.144	1.572
					RSA-10				
1.1	1.17	0.42	0.70	0.26	1.66	0.60	0.61	0.090	0.983
1.7	1.08	0.40	1.76	0.64	0.62	1.64	1.62	0.103	1.127
2.6	0.92	0.32	2.25	0.74	0.42	2.44	2.31	0.143	1.568
					RSA-5				
1.1	1.18	0.42	0.71	0.26	1.66	0.60	0.62	0.088	0.960
1.7	1.18	0.44	1.92	0.70	0.62	1.62	1.60	0.085	0.936
2.6	1.08	0.37	2.61	0.83	0.42	2.42	2.27	0.104	1.140

[a]W=41 220 kN; [b]V=0.091 W (3766 kN)

As permitted in NBCC 2010, the seismic analyses are performed independently along each orthogonal direction. All analyses are carried out using the ETABS computer program (CSI 2013) with a three-dimensional structural model to properly account for torsional effects. A typical model is shown in Fig. 3. All structures members were modelled as elastic frame elements. In the model, all beam-to-column joints, braces end connections and columns bases were considered as pinned connections. For the BRB members, the cross-sectional areas of the model frame elements were taken equal to 1.5 times the cross-sectional areas of the brace core, A_{sc}, to account for the larger axial stiffness present in the end protrusions and connections of the braces. For dynamic analysis, the structures translational masses in both orthogonal directions and polar mass moments of inertia were specified at every level. Further details on the structural model are given in section 4 of the paper.

The three methods of analysis available in NBCC 2010 for considering the effects of accidental torsion were applied for the design: (1) Equivalent Static Procedure with static in-plane torsional moments assuming a mass eccentricity of 10% of the building dimension (ESP); (2) Response Spectrum Analysis with static torsional moments based on 10% of the building dimension (RSA-10); and (3) Response Spectrum Analysis with the CM being displaced by *5%* of the building dimension (RSA-5). This resulted in 9 different structures: three values of B and three methods of analysis. As discussed previously, the RSA-10 method is permitted to be used in the NBCC 2010 for all three values of B covered in this study. The ESP is the simplest of the three methods but is not allowed for torsionally irregular buildings (B>1.7) located in high seismicity regions. As discussed later, the RSA-5 method typically leads to more cost-effective structures and is often preferred in practice for this reason. However, the method is not permitted when B>1.7. In this study, all analysis methods were considered, even if not permitted in the NBCC, to verify the appropriateness of current code provisions.

In the ESP, the earthquake load is linearly distributed at every level of the structure as a function of the storey elevations and seismic weights. Response spectrum analysis (RSA) is performed using the design spectrum S of Fig. 2. The results of the response spectrum analysis are

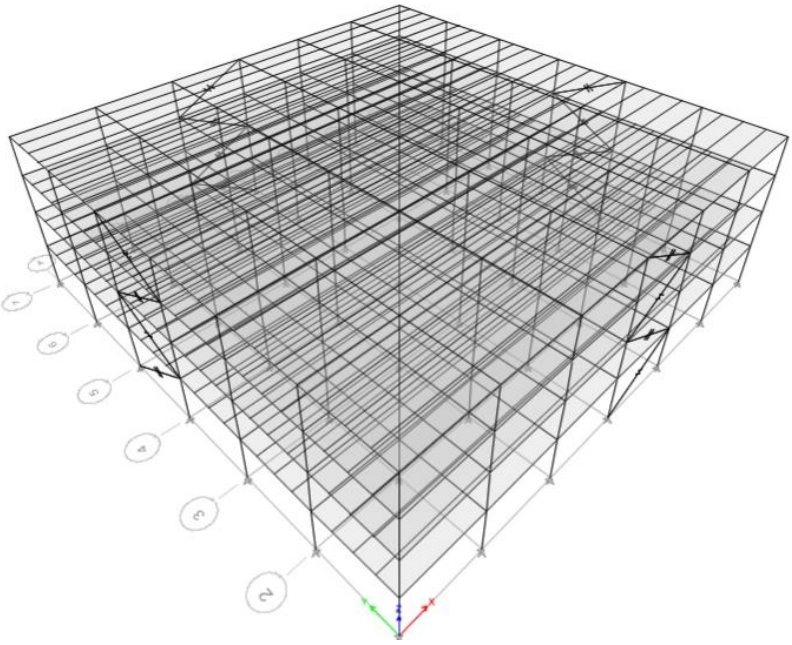

Fig. 3 Three-dimensional view of the structural analysis model (case B=1.1 shown)

then multiplied by $I_E/(R_dR_o)$ to account for importance, ductility and overstrength. After this adjustment, if the base shear from RSA, V_{RSA}, is less than V, all results must be multiplied by the ratio V/V_{RSA}. For regular buildings in torsion (B=1.1 and B=1.7), this calibration can be performed using 0.8 V, instead of V, as a reference. For all structures examined in this study, V_{RSA} was less than V, or 0.8 V, and all RSA results were calibrated accordingly. Hence, the structures with B=2.6 and designed using the RSA-10 and RSA-5 analysis methods were assigned seismic lateral load effects 20% higher than the equivalent structures with B=1.1 and 1.7.

In design, earthquakes effects must be combined with the effects of gravity loads including 100% of the dead loads (DL), 50% of the floor occupancy live loads (LL) and 25% of the roof snow load (SL). The applicable gravity loads are given in Fig. 1. Stability effects must also be included in seismic design. For the structures studied, however, P-delta effects were less than 10% of the effects of seismic lateral loads and, thus, have been ignored, as permitted in the NBCC. In the CSA S16 design standard for steel structures in Canada (CSA 2009), notional loads are also prescribed to account for initial out-of-plumbness and residual stress effects. Those loads were also omitted in this study as the structural analysis models did not include initial imperfections.

In accordance with capacity design principles, the buckling restrained bracing (BRB) members were designed first in the structures. In CSA S16, the factored axial resistance of the BRB's in tension (T_r) and compression (C_r) are respectively equal to $C_r=T_r=\phi A_{sc}F_{ysc}$, where ϕ=0.9 for steel, A_{sc} is the cross-section area of the brace core and F_{ysc} is the yield strength of the plate material used to fabricate the brace core. In this study, it was assumed that the brace cores were sized using the steel yield strength obtained from coupon tests of the plate material, as permitted in CSA S16, and F_{ysc}=280 MPa was therefore adopted, as commonly used in the industry. Once the brace design was finalized, beams and columns were then designed for the gravity loads effects plus the seismic effects amplified to reflect the probable yield strength of the BRB members. As specified in CSA S16, the brace probable tensile and compressive yield strengths are equal to ωP_{ysc} and $\beta\omega P_{ysc}$, respectively, where $P_{ysc}=A_{sc}F_{ysc}$, ω is the strain hardening adjustment factor and β is the friction adjustment factor. The factors ω and β were taken equal to 1.55 and 1.05, as obtained from full scale tests performed on Star Seismic BRB members having capacities similar to those considered in this study (Romero *et al*. 2007).

Table 2 gives the properties of the buildings designed using the three different methods of analysis. These properties were calculated for a symmetric-plan configuration. From left to right are the two first translational periods of vibration in the North-South direction ($T_{1,\,N\text{-}S}$ and $T_{2,\,N\text{-}S}$), the two first torsional periods of vibration ($T_{1\vartheta}$ and $T_{2\vartheta}$), the ratio of the uncoupled frequencies (Ω_R), and the ratios of the torsional periods on the translational periods ($T_\vartheta /T_{N\text{-}S}$). In the table, the last two ratios represent the total yield lateral resistance of the SFRS, V_y, normalized with respect to the seismic weight W (=41220 kN) and to the minimum design lateral earthquake load V (=0.091 W), where V_y is the base shear when the BRB members at the first level reach their axial yield strength P_{ysc}. A value of 1.11 (=1.0/ϕ) would therefore be obtained for a structure designed without accounting for accidental eccentricity.

In the NBCC, anticipated deflections including inelastic effects are obtained by multiplying the deflections from analysis by R_dR_o/I_E, thus assuming equal elastic and inelastic displacements. Storey drifts are then calculated using the maximum storey displacement at the extreme points of the structure, including torsional effects. The values are presented in section 5 of the paper. For all structures, the computed storey drifts were less than the limit of 2.5% times the storey height specified in the NBCC for buildings of the normal importance category; hence, drift limit did not govern the design.

For all structures, increasing the B factor by moving the bracing bents closer to the building geometric center increased the design force demand on the braced frames, which resulted in an increase of the overall SFRS lateral resistance V_y. Among all design methods, the RSA-5 approach required the lowest lateral resistance, regardless of the B factor. Conversely, ESP required the largest resistance. For the RSA methods and $B\leq1.7$, the ratio V_y/V can be smaller than 1.0 because scaling of the analysis results is performed with respect to 0.8 V. For B=2.6, the strength from ESP and RSA-10 are very close because the seismic effects in both cases are based on V, as opposed to 0.8 V for RSA-10 when B=1.1 and 1.7. For that same case (B=2.6), the required lateral strength from RSA-5 is much lower. For buckling restrained braced frames, lateral stiffness and strength are closely related and the variations in the translational periods can then be predicted from the variations in the lateral resistance. Hence, the structures with a larger B or designed using the ESP typically have relatively shorter translational periods. These structures therefore benefit from a greater lateral strength due to the code limit on the design period (2 T_{emp}). The ratio between the periods in the first two torsional modes to those in the corresponding two N-S translational modes increases with the B factor, reflecting the increasing torsional sensitivity as the torsional building stiffness reduces. For B=1.7 and 2.6, this ratio is greater than 1.0. For a given B value, the ratios between corresponding torsional and translational periods remain practically the same.

4. Analysis

4.1 Structure analysis and model

Time history analyses of the building structures were performed using the same structural models and analysis program as used in design. Linear response analyses were performed using the modal superposition time history technique with the eigenvalue properties and assuming 3% of critical damping in all modes. Nonlinear response analyses were performed using the direct integration analysis technique with the Newmark-Beta integration scheme. Proportional damping equal to 3% of critical in the first mode of vibration and in the mode for which 90% of the effective modal mass is participating, was adopted.

As indicated, in the structure model, the frame members representing the BRB members were assigned an equivalent cross-sectional area equal to 1.5 times the brace core area A_{sc} to reflect the greater axial stiffness present outside of the core segment (brace protrusions, end-connections and physical size of the beam-to-column joints). This 1.5 factor is representative of typical braces and core lengths that would be used in structures similar to those studied herein. The same structural models were used for both the linear and nonlinear analyses except that the BRB hysteretic response was considered for the latter. This was achieved by assigning deformation controlled (ductile) axial “frame hinges” to the BRB frame members and adding elastic “link” members acting in parallel to the brace elements.

The frame hinges were assigned a yield tensile resistance P_{ysc} equal to the brace core yield strength from design, assuming that the core steel yield strength in design was based on the measured properties of the core plate material employed for the fabrication of the BRB members, as commonly done in practice. In compression, the yield strength that was specified to the frame hinges was set equal to 1.05 P_{ysc}, consistent with the β value assumed in design. Isotropic hardening was also specified for the frame hinges such that the yield strength increases by 0.40 P_{ysc} when the structure reaches a storey drift equal to 2.0 times the anticipated storey drift, the deformation level prescribed for qualification testing as specified in the CSA S16 standard. As per NBCC 2010, the anticipated storey drift is equal to $R_dR_o\Delta_f$=4.8 Δ_f, where Δ_f is the storey drift under the design seismic loads, which corresponds to 0.9 (ϕ=0.9) times the storey drift producing yielding of the brace core, Δ_y. Hence, isotropic strain hardening parameters for the frame hinges were set to achieve 0.40 P_{ysc} additional brace resistance after reaching a storey drift of 8.64 Δ_y. Kinematic hardening response of the BRB members was modelled by adding an elastic “link” element with axial stiffness adjusted to give an extra 0.15 P_{ysc} capacity at the same test storey drift level. The total strain hardening in the model therefore corresponded to 0.55 P_{ysc} at a storey drift equal to 8.64 Δ_y, which corresponds to the ω value used in design. Figure 4(a) shows the cyclic inelastic response of the braces as modelled when subjected to the CSA S16 test protocol. A typical response from seismic analysis is shown in Fig. 4(b).

Because the structures have similar properties in both orthogonal directions, the response along only one direction (N-S) was examined. In all analyses, both orthogonal components of the ground motion records were applied simultaneously to the structures. Three conditions of eccentricity were considered for all time history analyses: no eccentricity in either direction (e=0), all masses displaced by 5% D_n in the E-W direction ($e_{E\text{-}W}$=0.05 D_n; $e_{N\text{-}S}$=0), and all masses displaced by 5% D_n in both directions ($e_{E\text{-}W}$=$e_{N\text{-}S}$=0.05 D_n). This 5% eccentricity corresponds to the accidental eccentricity associated to uncertainty in the position of the center of mass in the structure. Other sources of eccentricity such as the rotational component of the ground motions or the inherent variability in strength of the SFRS members have not been considered in the analyses. In all time history analyses, P-delta effects were considered with gravity loads corresponding to 100% DL, 50% LL and 25% SL being applied to the structures.

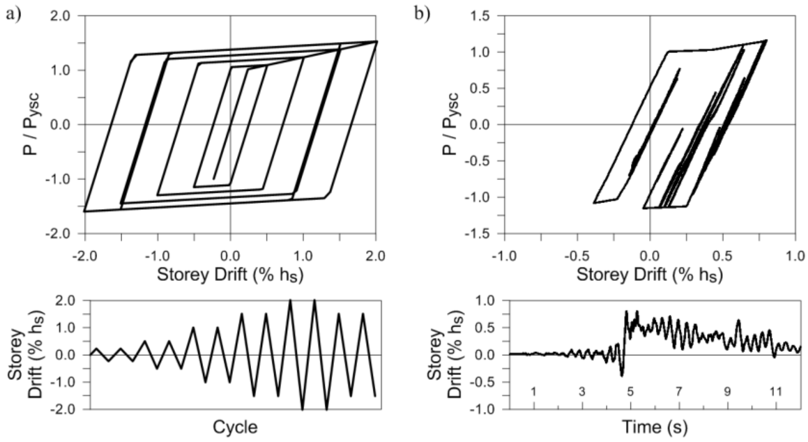

Fig. 4 Hysteretic response of the buckling restrained bracing members under: a) CSA S16 qualification test protocol, b) NGA#838 ground motion (first-storey brace, case B = 2.6 and RSA-5)

4.2 Ground motion time histories

The structures were subjected to a suite of 11 pairs of orthogonal horizontal ground acceleration components. The properties of the selected records are listed in Table 3. All motions were imported from the PEER Strong Motion Database (PEER 2010). The time histories were recorded at class *C* sites with the shear wave velocity parameter $V_{s,\,30}$ comprised between 360 m/s and 760 m/s, corresponding to the site condition assumed for the prototype buildings. The seismic events have magnitudes, M_w, and distances, R_{rupt}, that match the scenarios dominating the hazard in Vancouver, BC, Canada. For all record pairs, the first component given in the table was the one applied in the N-S direction of the prototype buildings. For each ground motion pair, the geometric mean of the 5% damped acceleration spectra was calculated, and a scaling factor, SF in Table 3, was applied such that the 84th percentile of the geometric mean spectral ordinates does not fall below the design spectrum in the period range defined as follows: a lower bound equal to the shortest period necessary to achieve a 90% mass participation, but not less than 0.2 *T*, and an upper bound equal to 2.0 *T*, but not less than 1.5 s, where the period *T* is the fundamental period of the structure for the direction being analyzed.

In this study, the structures were analyzed along the N-S direction and *T* was taken equal to the longest of the periods in the first translational mode in that direction, i.e., T=1.18 s. An upper bound of 2.36 s, greater than 1.5 s, was therefore adopted. For all structures, mass participation in the N-S direction exceeded 90% in the mode associated to the period $T_{2,\,N\text{-}S}$. The shortest of the three periods $T_{2,\,N\text{-}S}$ is 0.32 s, which is longer than 0.2 T=0.24 s. A lower bound of 0.32 s was therefore adopted for the scaling period range. The scaling factors are given in Table 3 and the 84th and 50th percentile geometric mean spectra for the suite of scaled records are plotted in Fig. 2. As shown, the 84th percentile spectrum satisfies the criterion except for the 1.75-2.36 s period range, where the demand is slightly below (10% on average) the design spectrum. This was done to avoid excessive demand at other periods. Note that if the upper bound of the period range was selected as 1.5 *T* (1.77 s), as specified, for instance, in ASCE 7-10, all 84th percentile spectral ordinates would be at least within 1% of the target spectrum. The 50th percentile spectrum matches the target spectrum in the intermediate period range but underestimates the design value at the lower and

Table 3 Properties of the selected ground motions

Event	M_w	Station	R_{rupt} (km)	$V_{s,30}$ (m/s)	NGA No.	Comp.	SF
1952 Kern County	7.4	Taft Lincoln School	39	385	15	21° & 111°	2.0
1989 Loma Prieta	6.9	APEEL 9 Crystal Spring Resort	41	450	736	227° & 137°	1.8
1989 Loma Prieta	6.9	Anderson Dam (Downstream)	20	489	739	250° & 340°	1.2
1989 Loma Prieta	6.9	Palo Alto - SLAC Lab	31	425	787	360° & 270°	1.0
1989 Loma Prieta	6.9	SF-Presidio	77	595	796	90° & 0°	1.4
1992 Landers	7.3	Barstow	35	371	838	90° & 0°	1.8
1994 Northridge	6.7	Castaic, Old Ridge Route	21	450	963	90° & 360°	0.6
1994 Northridge	6.7	LA - Brentwood VA Hospital	23	417	986	195° & 295°	1.5
1994 Northridge	6.7	LA - Temple & Hope	31	376	1005	180° & 90°	1.7
1994 Northridge	6.7	Moorpark - Fire Sta	25	405	1039	180° & 90°	1.4
1999 Hector Mines	7.1	Joshua Tree	31	379	1794	360° & 90°	1.4
Average	**6.9**		**34**	**431**			**1.4**

upper ends of the scaling period range.

When selecting the value of T for scaling, it was noted that the periods associated to pure torsion, $T_{1,\theta}$, exceeded the period $T_{1,N\text{-}S}$ for buildings with B=1.7 and 2.6, raising the question whether $T_{1,\theta}$ would be more appropriate than $T_{1,N\text{-}S}$ when establishing T. This issue is discussed when analyzing the results.

5. Analysis results

5.1 Typical time history responses

Figs. 5 and 6 present the time history responses for the three structures designed with the RSA-5 analysis method (B=1.1, 1.7 and 2.6) when subjected to one pair of ground motion records. The figures respectively give in-plane rotation time histories of the roof and first-storey diaphragms, as well as the maximum and average roof and first-storey drifts in the N-S direction. The figures can therefore be used to evaluate the efficiency of this analysis method in controlling the response of buildings with various torsional sensitivity levels. In these figures, the analyses were performed by displacing the CM of every storey by 0.05 D_n in the eastern direction, as discussed in section 4.1. The height h_s and h_n respectively correspond to the first storey (4.5 m) and the total (16.5 m) heights of the structures. For simplicity, only the N-S component of the ground motion is plotted for reference in the figures.

The diaphragm rotation and the maximum drifts generally increase with the torsional sensitivity factor, B. For the buildings with B=1.1 and B=1.7, the maximum drifts and the rotations reach a steady state after the strong ground motion segment of the records, unlike the building with B=2.6 where the maximum response keeps increasing in time. Furthermore, the gap between the average and the maximum storey drift responses increases with the B factor. However, this gap tends to stay constant for buildings with B=1.1 and B=1.7 while it progressively increases for the building with *B*=2.6. For these structures, the RSA-5 analysis method is less effective in controlling the maximum response in the case of a highly torsionally sensitive building.

For the ground motion and time window selected in Figs. 5 and 6, the maximum drifts in the N-S direction remain below the 2.5% h_s limit specified in the NBCC 2010 for buildings of the normal importance category. However, this was not always the case. For example, structures with *B*=2.6 experienced excessive lateral displacements (5-10% h_s), near structural collapse, when the RSA-5 analysis method was used in the design process. In Fig. 7, the maximum roof drifts and the roof diaphragm rotations are presented for the three buildings with *B*=2.6 designed according to the three NBCC analysis methods. The response of the building designed with the RSA-5 analysis method increases progressively, which suggests that this method is not efficient for controlling the response of torsionally sensitive buildings. Conversely, the same structure with *B*=2.6 exhibited satisfactory performance when designed using the ESP and RSA-10 analysis methods. These observations support the current NBCC 2010 provisions regarding the applicability of the RSA-5 analysis method for the seismic design of buildings: as explained earlier, this method is only permitted for buildings with *B*<1.7.

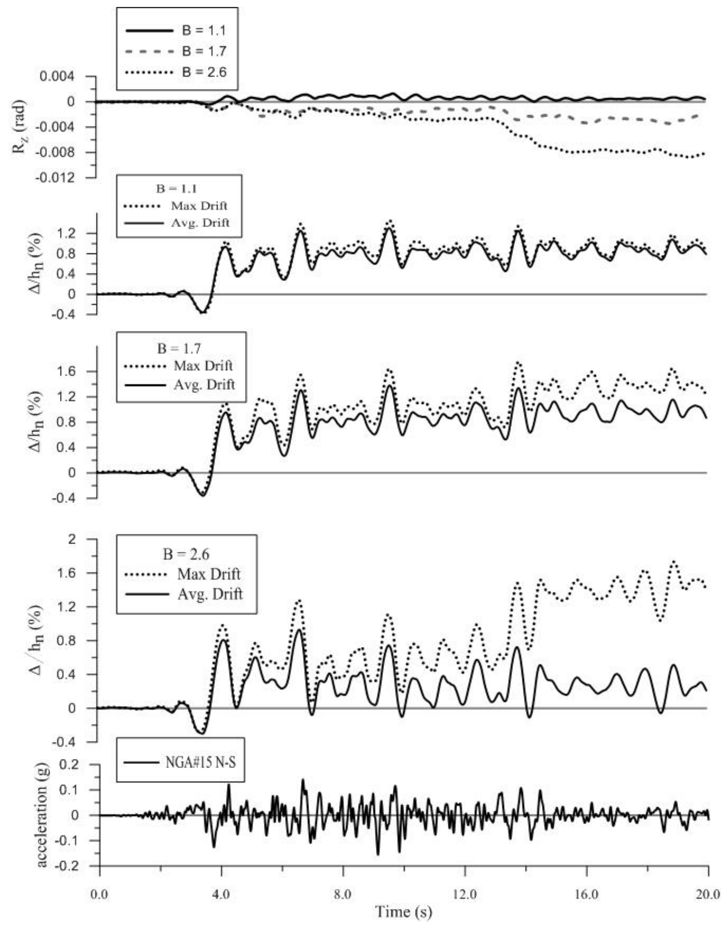

Fig. 5 Time history response of the roof drift and diaphragm rotation for the structures designed with the RSA-5 analysis method

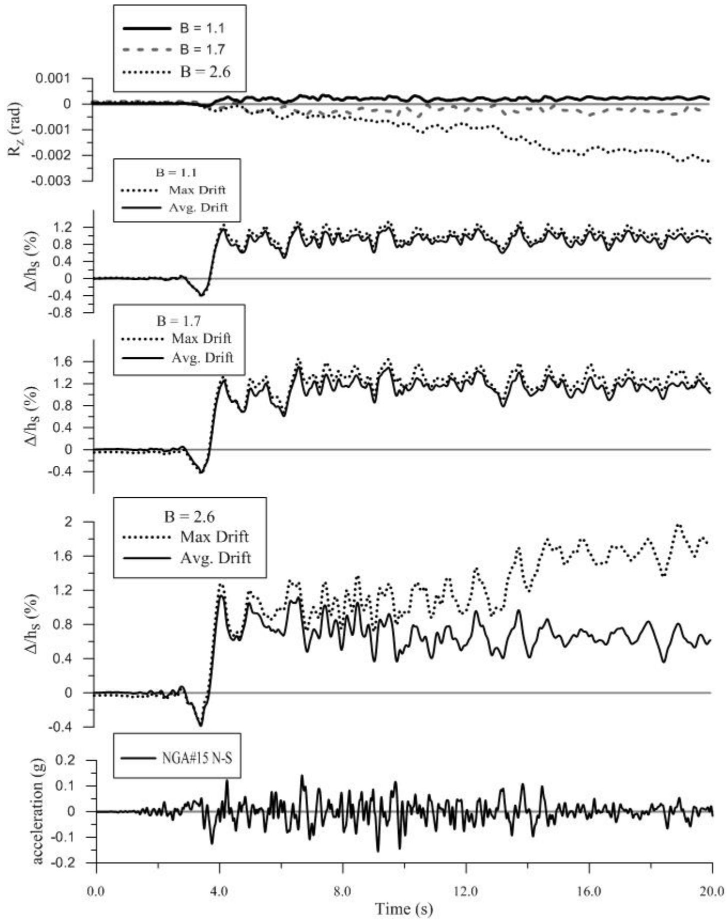

Fig. 6 Time history response of the first-storey drift and diaphragm rotation for the structures designed with the RSA-5 analysis method

When comparing the results obtained for the first storey and the roof level in Figs. 5 and 6, it is observed that the trends are very similar at both locations. Although the rotations of the diaphragms are more important at the roof level than at the first storey, the storey drifts are of the same order of magnitude under this selected ground motion. This indicates that higher modes have limited effects on the response of the four-storey buildings studied in this paper.

5.2 Statistics of peak storey drifts

Table 4 gives the 84th percentile values of the peak storey drifts obtained from the response history analyses of all structures subjected to the eleven two-component historical records. Only the values obtained in the N-S direction are presented; similar results and conclusions would be obtained by considering the E-W direction because of the symmetry of the structures. The 84th percentile values are given in Table 4 as the 84th percentile spectrum was found to generally match or exceed the design spectrum in Fig. 2. All storey drift values in the table are given as a percentage of the respective storey heights. For all cases, results are presented respectively for the

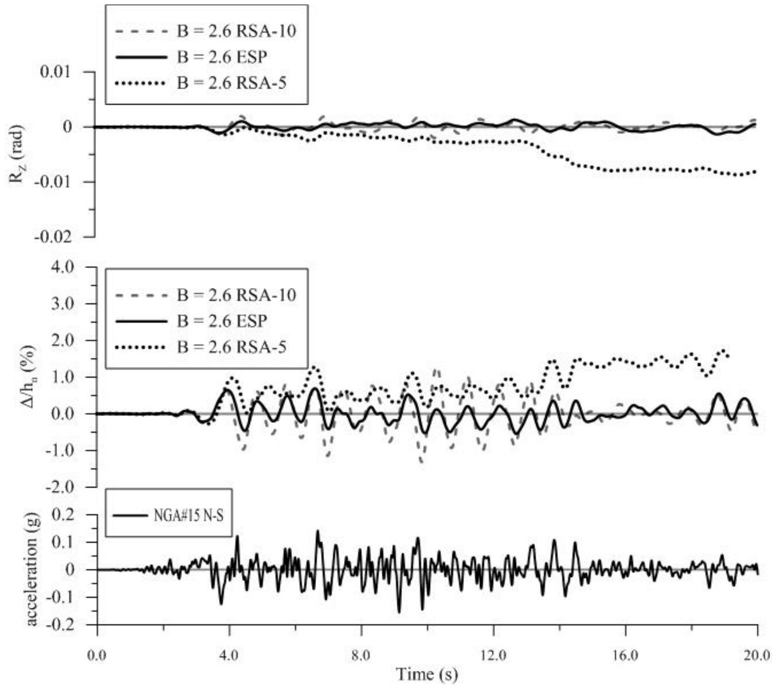

Fig. 7 Time history response of the roof maximum drift and diaphragm rotation for structures with *B*=2.6

two extremities of the buildings, and that, for the three conditions of eccentricity presented earlier in section 4.1. The values in brackets are the storey drifts obtained from the linear modal time history analyses. These are given for comparison purposes. The anticipated storey drifts including inelastic effects, as predicted in the design process, are also included in the table for comparison. These design values are the maximum values over the building width.

As discussed, excessive displacements (5-10% h_s) were obtained under a certain number of ground motions for the buildings with *B*=2.6 designed with the RSA-5 analysis method. It is noted that this behaviour occurred even if scaling of the analysis results was performed with respect to 100% *V*, instead of 80% *V*, as required for torsionally sensitive structures. Although the structure remained stable in the analyses, these displacements exceed the acceptable limit of the NBCC 2010 (2.5 % h_s) and some buildings were close to the collapse state. As demonstrated in Fig. 7, the RSA-5 analysis method was not adequate to control the response of the highly torsional sensitive buildings with *B*=2.6 examined in this study. These findings are in agreement with the current NBCC 2010 provisions where the RSA-5 analysis method is not permitted for buildings having a torsional sensitivity ratio, *B*, greater than 1.7.

No structural collapse was observed for all other *B* factor and analysis method cases considered in this study. For all these other cases, good performance could be achieved. Even when considering eccentricities in both the E-W and N-S directions, the peak storey drifts from nonlinear analysis all remained below the limit specified in the NBCC 2010 for buildings of the normal importance category ($\Delta/h_s \leq 2.5\%$). This suggests that the RSA-5 method was adequate for *B*=1.1 and 1.7 and that the ESP and RSA-10 methods gave satisfactory performance for all three values of *B* examined in this study. Results for these cases are discussed in greater details in the following paragraphs.

By comparing the results obtained from nonlinear analyses for the three eccentricity cases considered in the analyses, i.e., e=0, e=0.05 D_n in the E-W direction and e=0.05 D_n in both orthogonal directions, the peak storey drifts are generally smallest for the case e=0 and largest for the case where accidental eccentricity is considered in both directions. The differences generally increase as B is increased. This suggests that ADE should be considered in both orthogonal directions for the design of structures exhibiting torsional sensitivity. The analysis method does not seem to influence significantly this effect. For the cases where e=0, the storey drifts obtained from the nonlinear response history analyses are not the same at the two building extremities although the structures are perfectly symmetric. The differences are due to the fact that the ground motions are applied simultaneously in both orthogonal directions, causing unequal demands on the two parallel bracing bents.

The peak storey drifts obtained for the east and west sides of the buildings are compared to evaluate the influence of torsional sensitivity on the distribution of the displacement demand. For the buildings with B=1.1, the maximum storey drifts occur on the East side. Conversely, for B=1.7 and 2.6, the displacement demand is generally larger on the West side, i.e. on the side opposite to the accidental eccentricity considered in the analysis. Again, the observed tendencies are not affected by the analysis method used in design. With some exceptions, particularly in the first storey, the same observations can be made by examining results from elastic time history analyses. These results suggest that the displacement demand is generally greater on the side located closer to the CM for symmetrical torsionally stiff structures while the demand is higher on the side located farther from the CM for symmetric torsionally sensitive structures. These results are in agreement with the statement from Basu *et al*. (2014) that a ratio of 1.0 between the first torsional period and the first translational period delimitates torsionally sensitive and non-sensitive structures. In Table 2, all structures with B=1.7 and 2.6 have a period ratio greater than 1.0 and, according to this criteria, would therefore qualify as torsionally sensitive (see Table 2). The observation for the more torsionally sensitive structures is in agreement with the results obtained in the study by Erduran and Ryan (2011) for three-storey steel structures with peripheral braced frames.

By comparing the maximum storey drifts predicted in the design phase (ESP, RSA-10 and RSA-5) to those obtained on the East and West sides from the nonlinear analyses with e_{E-W}=0.05D_n and e_{N-S}=0, the former values are always smaller for B=1.1. The differences are more pronounced when using the RSA method with average ratios of 0.90 for the ESP method and 0.75 for the two RSA approaches. For B=1.7 and 2.6, the design values generally exceed the results from nonlinear time history analyses for the structures designed with the ESP and RSA-10 analysis methods. All drifts predicted with the RSA-5 method underestimate the nonlinear time history results for B=1.1, B=1.7 and B=2.6. These trends are obviously more pronounced when comparing design predictions to drifts obtained with accidental eccentricity considered in both directions in the analysis. This suggests that the NBCC approach for estimating drifts is non-conservative for torsionally regular structures while being conservative for more torsionally sensitive structures if the ESP and RSA-10 methods are adopted.

By comparing the storey drifts obtained from nonlinear analysis for the buildings designed with the ESP to those designed according to the RSA-10 and the RSA-5, the drifts are generally larger for the buildings designed according to the two dynamic analyses methods. However, the drifts of the first storey and the roof are in many cases greater for the buildings designed with the ESP. This suggests that the ESP may not provide appropriate vertical distribution of the resistance within the

Table 4 N-S peak storey drifts (84th percentile) obtained from response history analyses[1] in % of h_s

B	Storey	Design	East Side e=0	East Side $e_{E\text{-}W}$=0.05D_n	East Side $e_{E\text{-}W}$=$e_{N\text{-}S}$=0.05D_n	West Side e=0	West Side $e_{E\text{-}W}$=0.05D_n	West Side $e_{E\text{-}W}$=$e_{N\text{-}S}$=0.05D
ESP								
1.1	4	1.14	1.35 (1.46)	1.43 (1.54)	1.38 (1.50)	1.34 (1.46)	1.22 (1.32)	1.14 (1.32)
	3	1.09	1.08 (1.14)	1.15 (1.23)	1.15 (1.21)	1.09 (1.14)	1.03 (1.06)	1.01 (1.12)
	2	0.98	0.94 (0.98)	1.03 (1.05)	1.09 (1.06)	1.00 (0.98)	0.89 (0.91)	1.11 (0.94)
	1	0.80	0.97 (0.88)	0.91 (0.95)	1.31 (0.96)	0.96 (0.88)	0.88 (0.83)	1.18 (0.83)
1.7	4	1.58	1.30 (1.37)	0.93 (1.18)	1.20 (1.41)	1.25 (1.37)	1.53 (1.85)	1.72 (1.84)
	3	1.55	0.97 (1.17)	0.85 (1.03)	1.05 (1.12)	0.94 (1.17)	1.14 (1.40)	1.33 (1.58)
	2	1.43	0.87 (1.01)	0.83 (0.88)	1.20 (1.04)	0.92 (1.01)	0.93 (1.36)	1.12 (1.35)
	1	1.19	1.14 (0.93)	1.07 (0.78)	1.76 (0.93)	0.85 (0.93)	0.94 (1.21)	1.26 (1.26)
2.6	4	2.62	1.12 (1.50)	0.98 (1.28)	1.09 (1.40)	1.14 (1.50)	1.37 (1.91)	1.57 (1.85)
	3	2.45	0.91 (1.18)	0.92 (1.05)	1.04 (1.18)	0.94 (1.18)	1.13 (1.48)	1.38 (1.53)
	2	2.09	0.93 (0.94)	0.98 (0.85)	1.08 (0.94)	0.77 (0.94)	0.92 (1.17)	1.28 (1.17)
	1	1.65	0.91 (0.81)	1.14 (0.74)	1.25 (0.84)	0.82 (0.81)	0.87 (1.04)	1.32 (1.04)
RSA-10								
1.1	4	0.98	1.35 (1.54)	1.35 (1.64)	1.44 (1.67)	1.31 (1.54)	1.21 (1.38)	1.30 (1.43)
	3	1.03	1.28 (1.51)	1.34 (1.61)	1.45 (1.62)	1.23 (1.51)	1.16 (1.38)	1.33 (1.40)
	2	0.97	1.15 (1.19)	1.20 (1.27)	1.50 (1.27)	1.15 (1.19)	0.98 (1.10)	1.48 (1.17)
	1	0.83	1.01 (1.03)	1.05 (1.11)	1.58 (1.09)	1.18 (1.03)	1.04 (0.95)	1.46 (0.94)
1.7	4	1.45	1.33 (1.45)	1.20 (1.07)	1.25 (1.39)	1.31 (1.45)	1.56 (1.85)	1.78 (2.00)
	3	1.54	1.19 (1.27)	1.03 (1.04)	1.14 (1.38)	1.21 (1.27)	1.40 (1.55)	1.60 (1.55)
	2	1.49	1.05 (1.12)	0.93 (0.94)	1.28 (1.14)	1.12 (1.12)	1.22 (1.44)	1.32 (1.46)
	1	1.27	1.10 (0.99)	1.05 (0.79)	1.33 (0.95)	0.98 (0.99)	1.09 (1.31)	1.08 (1.28)
2.6	4	2.35	1.03 (1.35)	0.97 (1.24)	0.98 (1.30)	0.93 (1.35)	1.43 (1.74)	1.47 (1.77)
	3	2.32	0.93 (1.16)	0.97 (1.08)	1.00 (1.20)	0.92 (1.16)	1.39 (1.49)	1.45 (1.56)
	2	2.05	0.93 (0.95)	1.15 (0.87)	1.11 (1.00)	0.85 (0.95)	1.06 (1.19)	1.42 (1.20)
	1	1.67	1.05 (0.80)	1.20 (0.72)	1.29 (0.78)	0.75 (0.80)	0.90 (1.05)	1.24 (1.02)
RSA-5								
1.1	4	0.95	1.32 (1.50)	1.25 (1.61)	1.36 (1.62)	1.21 (1.50)	1.15 (1.39)	1.24 (1.44)
	3	0.98	1.21 (1.51)	1.29 (1.62)	1.37 (1.62)	1.15 (1.51)	1.10 (1.39)	1.25 (1.40)
	2	0.96	1.10 (1.22)	1.15 (1.30)	1.47 (1.31)	1.11 (1.22)	0.96 (1.12)	1.39 (1.19)
	1	0.81	1.06 (1.07)	1.11 (1.16)	1.47 (1.14)	1.11 (1.07)	1.02 (0.98)	1.45 (0.93)
1.7	4	1.03	1.21 (1.49)	1.13 (1.27)	1.29 (1.49)	1.26 (1.49)	1.64 (1.97)	1.66 (1.72)
	3	1.09	1.23 (1.48)	1.22 (1.23)	1.30 (1.46)	1.32 (1.48)	1.56 (1.91)	1.53 (1.75)
	2	1.09	1.04 (1.24)	1.08 (1.04)	1.14 (1.23)	1.12 (1.24)	1.25 (1.66)	1.32 (1.67)
	1	0.97	1.17 (1.06)	1.33 (0.87)	1.15 (0.98)	1.35 (1.06)	1.49 (1.43)	1.31 (1.53)
2.6	4	1.15	1.43 (1.56)	1.46 (1.36)	1.40[2] (1.51)	1.27 (1.56)	1.67 (2.03)	1.72[2] (2.04)
	3	1.15	1.48 (1.39)	1.66 (1.18)	1.47[2] (1.32)	1.28 (1.39)	1.81 (1.69)	1.57[2] (1.60)
	2	1.06	1.62 (1.03)	1.75 (0.94)	1.51[2] (1.04)	1.17 (1.03)	1.91 (1.29)	1.56[2] (1.35)
	1	0.85	1.79 (0.92)	1.85 (0.84)	1.38[2] (0.89)	1.08 (0.92)	1.98 (1.17)	1.52[2] (1.17)

[1]Linear modal analysis results are given in brackets ( )

[2]For two ground motions, storey drifts >5% h_s

braces of the structure. This shortcoming of the ESP is more pronounced for the highly torsional sensitive buildings (B=2.6), which supports the current NBCC provisions that preclude the use of the ESP for buildings with B>1.7 when located in moderate and high seismicity regions. In this study, the building with B=2.6 designed with the ESP showed good performance, with drifts below the code limit. However, this can be partly attributed to the overstrength resulting from the limitation on the building period T_a to be used in design. This overstrength may not be present in other structures and adequate performance should be verified for other prototype structures before suggesting a relaxation of current NBCC limitations on the use of ESP for torsionally sensitive structures.

The comparison of the demand from linear and nonlinear time history analyses is used to assess the possible influence of torsional sensitivity on the magnitude and distribution of inelastic demand in structures. When accidental eccentricity is not included in the response history analyses (e=0), the storey drifts obtained from the linear analyses are greater than those obtained from the nonlinear analyses except at the bottom storey of the majority of the structures studied. Similar response is obtained for structures designed with the ESP and RSA-10 methods. It is less pronounced when the RSA-5 method was used. This indicates that BRBFs possess sufficient overall energy dissipation capacity to control the inelastic drift demand but that demand was not distributed uniformly over the frame height for most of the four-storey braced frames studied herein, even in absence of accidental eccentricity. This behaviour becomes more pronounced when accidental eccentricity is included in the analysis, with a larger number of storeys with larger inelastic demand and larger nonlinear to linear response ratios compared to the case with e=0. Higher differences between drift estimates from nonlinear and linear time history analyses are also generally observed for B values larger than 1.1. This suggests that torsionally sensitive structures are more prone to concentration of inelastic demand along their height. The analysis method used in design does not seem to affect this tendency.

7. Conclusions

A parametric study was performed to investigate the effects of accidental design eccentricity on the seismic response of steel building structures. Four-storey prototype structures with buckling restrained braced frames in both orthogonal directions were studied. The structures were designed according to the 2010 Canadian seismic code provisions. All structures had symmetrical plan geometries with no inherent eccentricity between the center of stiffness and the center of mass. The position of the bracing bents was varied to evaluate the seismic response of structures having three different levels of torsional sensitivity: B=1.1, 1.7 and 2.6. The three different seismic analysis methods available in the 2010 National Building Code of Canada were used in the design: equivalent static procedure with static torsional moments based on 10% of the building dimension (ESP); response spectrum analysis with static torsional moments based on 10% of the building dimension (RSA-10); response spectrum analysis with the CM being displaced by 5% of the building dimension (RSA-5). The structures were subjected to a set of eleven two-component historical records. The following main conclusions can be drawn:

The overall lateral resistance of the prototype structures increases when their torsional sensitivity (parameter B) increases. For a given B value, the ESP design resulted in the largest lateral resistance while the weakest structures were those designed with the RSA-5 method.

For the low-rise buildings studied herein, the fundamental mode of vibration was purely torsional for buildings with *B* equals to 1.7 and 2.6, regardless of the design method used. For these structures, the inelastic demand on the yielding seismic force resisting system (SFRS) components was affected by the building rotational response due to its fundamental torsional mode, which suggests that this period should be considered when establishing the range of periods over which ground motion records for analysis are scaled.

For structures with *B*=1.1, maximum lateral displacements occurred on the same side as the accidental eccentricity whereas the opposite was observed for the structures with *B*=1.7 and 2.6. This suggests that *B*=1.7 may not be sufficiently stringent to distinguish between torsionally sensitive and non-sensitive structures. When *B* is close to 1.7, the frequency ratio Ω_R may also be considered to verify torsional sensitivity.

The RSA-10 and the ESP analysis methods gave satisfactory seismic structural response for all *B* values, including *B*=2.6. For these two methods, the drifts predicted in the design phase were conservative for buildings with *B*=1.7 and *B*=2.6. This suggests that the ESP could be adequate for highly torsionally sensitive buildings, contrary to current NBCC provisions. Additional studies based on incremental dynamic analysis should be performed on additional prototype structures to verify if such a relaxation could lead to satisfactory performance.

Occurrences of excessive storey drifts (5-10% h_s) were observed when using the RSA-5 analysis method for a structure with *B*=2.6, which confirms current code provisions that this method is not appropriate for highly torsionally sensitive structures.

In this study, only the three design approaches for accidental torsion that are included in the 2010 NBCC have been considered. Also, only structures with symmetrical SFRS and mass arrangements have been examined. Alternative methods proposed in other codes as well as buildings with unsymmetrical structural and mass properties should be examined in future studies. Further research is also required to determine whether the conclusions drawn from this study can be extrapolated to building structures for which higher modes have a noticeable effect on the seismic response. Finally, this study was limited to translational ground motion effects; code provisions for accidental torsion should also be validated for rotational ground motion demands (Falamarz-Sheikhabadi and Ghafory-Ashtiany 2012).

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