Seismic Torsional Response on Steel BRB Frames: Inelastic modelling and code design provisions

Charles Bussières[1], Robert Tremblay[2], Pierre Léger[2]

[1] M.Eng Student, Department of Civil, Geological and Mining Engineering, Polytechnique Montreal - Montreal, QC, Canada.
[2] Professor, Department of Civil, Geological and Mining Engineering, Polytechnique Montreal - Montreal, QC, Canada.

ABSTRACT

The adequacy of current code design provisions for accidental torsion in symmetrical buildings is examined for four-storey steel buckling restrained braced frames (BRBFs). The structures were designed according to provisions of the 2010 National Building Code of Canada (NBCC) but accidental torsion provisions from ASCE7-10, EC8 and NCh433 were also considered. Different levels of torsional sensitivity (B) were studied by changing the in-plan location of the braced bays and the building geometry. The equivalent static force procedure (ESP) and response spectrum analysis (RSA) were also examined. Nonlinear Response History Analyses (NRHA) were performed using the Fast Nonlinear Analysis (FNA) strategy available in the ETABS computer program. Different hysteretic models were first examined and compared for the BRB members and a simple bilinear model, calibrated against more refined models, was selected for NRHA. The study confirmed the importance of considering accidental torsion in the design of symmetrical buildings, especially for torsionally sensitive structures ($B > 1.7$). Regardless of the analysis methods used in design, maximum storey drifts increase with the building torsional sensitivity. For torsionally sensitive buildings, RSA with the center of mass displaced by 5% of the building dimension (D_n) is the less effective for controlling torsional response. The RSA with static torques due to an eccentricity corresponding to 0.05 D_n was found adequate for all torsional sensitivity levels considered. The ESP with 0.05 D_n is adequate for torsionally regular buildings ($B \leq 1.7$). The eccentricity must be doubled for the other buildings.

Keywords: buckling restrained braced frames; accidental design eccentricity; torsional sensitivity; hysteresis models

INTRODUCTION

Torsional motions under seismic events are essentially due to the inherent eccentricity between the center of mass (CM) and the center of lateral resistance (CR). However, rotational ground motion components and uncertainty in the distribution of mass, stiffness and yield strength of structural components may also contribute to torsional response (Chopra and De la Llera [1]). In codes such as the National Building Code of Canada (NBCC) [2], ASCE7-10 [3], Eurocode 8 (EC8) [4] or NCh433 [5], these additional sources of torsion are considered in design by means of the Accidental Design Eccentricity (ADE). Different ADE values are specified in these codes and the definition of torsionally regular (insensitive) or irregular (sensitive) buildings also varies between codes.

For steel braced frames, Erduran and Ryan [6] observed that ADE can cause yielding to occur in one bracing bay, leading to a "flexible" building side and 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 design. Roy et al. [7] examined the response of four-storey symmetrical square buildings with steel buckling restrained braced frames (BRBFs) designed in accordance with 2010 NBCC. Three bracing bay arrangements were considered to vary the sensitivity to torsion, from stiff structures, with perimeter braced frames, to torsionally flexible core-type structures. The structures were assigned 5% mass eccentricity and subjected to bi-directional ground motion records. In the analyses, mass eccentricity was assigned in both orthogonal directions as this led to larger lateral displacements and more pronounced differences for the more torsionally sensitive structures. The study confirmed the findings by Erduran and Ryan [6] as larger displacements occurred on the same side as the accidental eccentricity for the torsionally stiffer structures whereas the opposite was observed for the torsionally more flexible buildings. In their study, Roy et al. [7] used a detailed numerical model to accurately reproduce the isotropic and kinematic hardening of the buckling restrained bracing members while accounting for P-delta effects. The model was however very computationally expensive, which limited the number of cases that could be investigated.

This paper reports on an extension of the study by Roy et al. [7] in which additional BRB hysteretic models are used in combination with the Fast Nonlinear Modal Superposition time history analysis technique available in the ETABS computer program (CSI 2015) to investigate further the torsional seismic response of four-storey BRBFs. Additional building geometries were also considered to examine five different torsional sensitivity levels (B =1.1, 1.4, 1.7, 2.6 and 2.8). Design provisions for accidental torsion in ASCE 7–10 (US) [7], EC-8 (Europe) [4] and NCh433 [5] (Chile), were also examined as potential alternative to methods prescribed in the NBCC [2]. Three analysis methods were used for design: Equivalent Static Procedure with in-plane static torsional moments (ESP-X), Response Spectrum Analysis with in-plane static torsional moments (RSA-X), and Response Spectrum Analysis with the CM being displaced (RSA^{CM}-X), where X is the accidental eccentricity expressed in percentage of the building plan dimension perpendicular to the seismic loading. This research pursues the following main objectives: (1) evaluation of the influence of torsional sensitivity on the response of multi-storey symmetrical steel buildings with representative geometry and seismic force resisting system (SFRS) subjected to strong ground motions, (2) evaluation of the seismic provisions of the NBCC 2010 relative to torsion, (3) evaluation and comparison of various methods prescribed in other codes to account for ADE, and (4) provide practicing engineers and researchers with recommendations for performing NRHA easily and with computational efficiency.

CODE PROVISIONS AND DESIGN

Seismic codes torsionnal provisions

In the NBCC 2010, buildings with rigid floor diaphragms are considered as having a torsional irregularity when $B_x = \delta_{max}/\delta_{avg} > 1.7$, where δ_{max} and δ_{avg} are the maximum and average storey displacements at the extreme points of the structure at level x due to the equivalent static seismic loads applied at a distance ±0.10 D_n from the CM, and D_n is the building dimension at level x perpendicular to the direction of the earthquake load. B is taken as the maximum value among all storeys and along both orthogonal directions. A symmetrical building may therefore be irregular in torsion if it has relatively low torsional stiffness. Torsional irregularity impacts the analysis method and the method used to account for ADE. For buildings with no torsional irregularity, the ESP is allowed for structures 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 RSA may be performed introducing the ADE by shifting the CM by ±0.05 D_n. For torsionally irregular structures, RSA is required and ADE is accounted for by means of static in-plane 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 above methods, the same sign can be taken over the building height for the eccentricity. Torsion irregularity may also influence the design when using the RSA method as the analysis results must be scaled such that the base shear from RSA is not less than 100% of the base shear from ESP for torsionally irregular buildings, and not less than 80% of the base shear from ESP for torsionally regular buildings.

Table 1 compares the analysis methods and requirements for ADE that are specified in 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 conditions of torsional irregularity. The factor A_x is to counter the possible dynamic shift of the CR due to unequal yielding of the SFRS elements and the resulting greater torsional rotations. In ASCE 7, torsional irregularity exists if the B factor computed with A_x = 1.0 is larger than 1.2. If B exceeds 1.4, the structure is considered as having extreme torsional irregularity. Eurocode 8 provisions are similar to those in ASCE 7 except that ADE is not amplified in the ESP. If RSA is used, ADE in EC8 is considered only through static in-plane torsional moments. In the NCh433 Chilean 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 are in the same direction at every level in multi-storey buildings (De-la-Colina et al. [8]).

Table 1. Comparison of code provisions for in-plane torsional moments due to accidental eccentricity

	Building Code			
Analysis Method	Canada NBCC 2010	United States[1] ASCE 7-10	Europe EC8 2004	Chile NCh433 2009
ESP	±0.10 D_n	(±0.05 D_n) A_x	±0.05 D_n	±0.10 D_n (Z/h)
RSA	±0.10 D_n or CM shifted by ±0.05 D_n	(±0.05 D_n) A_x or CM shifted by ±0.05 D_n	±0.05 D_n	±0.10 D_n (Z/h) or CM shifted by ±0.05 D_n

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

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

Building description

The five four-storey symmetrical steel office buildings with an equivalent floor area shown in Fig. 1 were examined in this study. Four of them have a square plan configuration and one is rectangular in shape. The structures are symmetric, i.e., no inherent eccentricity exists between their CR and CM. The lateral resistance in each orthogonal direction is provided by two buckling restrained braced frames (BRBFs). The location of the braced bays is varied to create five levels of torsional sensitivity, B. BRBFs were selected for this study because of their high ductility and limited overstrength, which makes the system prone to large inelastic excursions in case of excessive seismic demand resulting from inadequate vertical or horizontal distribution of the lateral resistance. Therefore, they are ideal candidates to study design provisions for inelastic seismic torsional response. As shown in Fig. 1, single diagonal bracing configuration is adopted for all braced frames. The design gravity loads are also given in the figure. The roof and floors consist of a 76 mm deep steel deck with a 63 mm thick concrete slab topping forming rigid horizontal diaphragms.

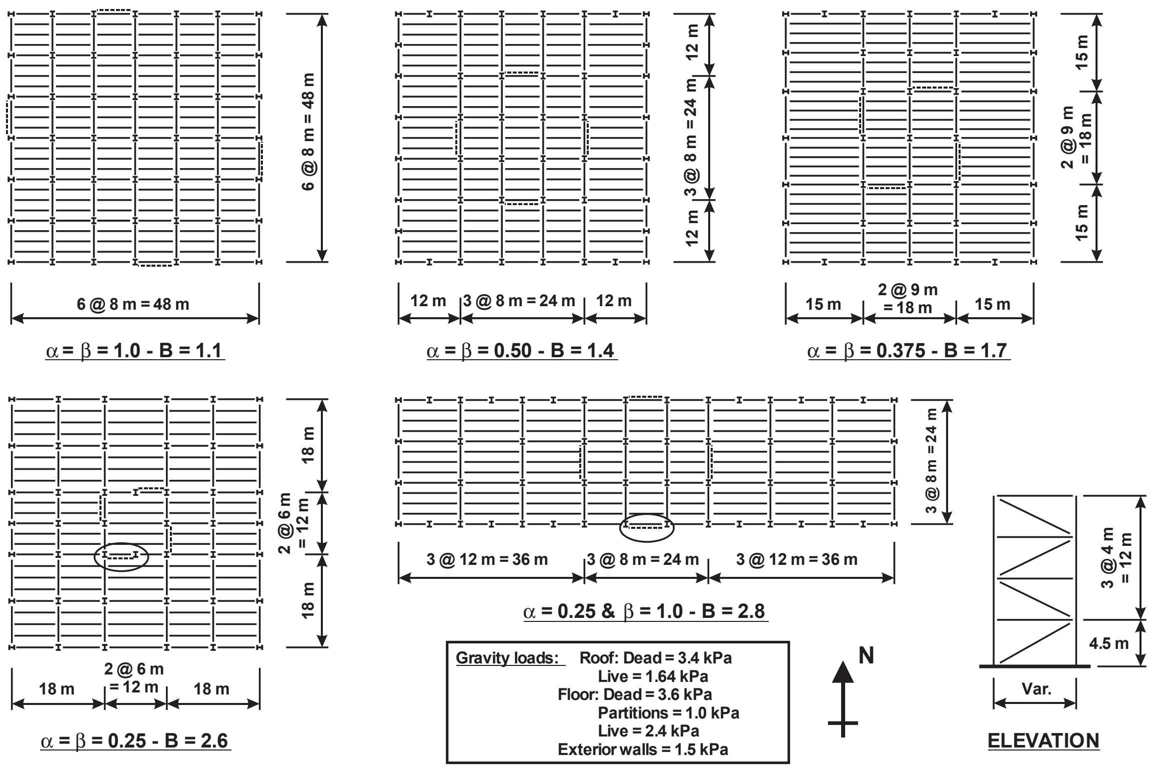

Figure 1. Geometry and design gravity loads of the buildings (CM and CR located at the geometric center of the buildings).

As shown, in each structure, all four braced frames were positioned at the same distance from the center of the building to obtain a doubly-symmetrical SFRS. For the four square buildings, the accidental eccentricity is the same in both directions and all structures therefore have four identical braced frames, regardless of the analysis method used in design. For this reason, the B factors are also the same in both directions and only depend on the distance separating the parallel braced frames. In Fig. 1, parameters α and β represent the ratios of the braced frames spacing to the building widths in the E-W and N-S directions, respectively. As shown, these ratios were varied and their values adjusted to obtain different cases for torsional sensitivity ranging from torsionally stiff buildings (B = 1.1) to torsionally sensitive core type buildings (B = 2.6). For the rectangular building, the larger E-W building dimension resulted in larger accidental torsional moments when the seismic loads were applied in the N-S direction. This required stronger and stiffer braced frames in that direction compared to the E-W braced frames. Moreover, the larger accidental torsional moments associated to N-S seismic loading induce larger in-plane rotations that combine with the larger distance between the east and west building edges to give larger B ratios. For the structure studied, B = 2.8 in the N-S direction compared to B = 1.1 in the E-W direction.

In this study, 7 different methods of analysis were considered for the design of each of the 5 buildings (35 cases in total). The first three methods are those specified in NBCC and given in Table 1, i.e. (i) ESP-10 (ESP with static torsional moments due to ADE = ±0.10 D_n), (ii) RSA-10 (RSA with static torsional moments due to ADE = ±0.10 D_n) and (iii) RSA^{CM}-05 (RSA with CM displaced by ±0.05 D_n). The 4th method, referred to as (iv) ESP-05-A_x, is prescribed in ASCE 7 (ESP with static torsional moments due to ADE = ±0.05 D_n augmented by the amplification factor A_x). The next two analysis methods are prescribed in EC8, i.e., (v) ESP-05 (ESP with static torsional moments due to ADE = ±0.05 D_n) and (vi) RSA-05 (RSA with static torsional moments due to ADE = ±0.05 D_n). The 7th method, referred to as (vii) ESP-10-Z/h, is prescribed in the NCh433 code (ESP with static torsional moments due to ADE = ±0.10 D_n and factored by the ratio Z/h). ASCE 7, EC8, and NCh433 methods are considered herein to evaluate if they can enhance the torsional response of the structures designed according to the NBCC.

Building design

In the NBCC 2010, the minimum design lateral earthquake force, V, is given by:

$$V = S(T_a)M_V I_E W/(R_d R_o) \tag{1}$$

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 mode effects, I_E is the importance factor, W is the seismic weight corresponding to the dead load plus 25% of the roof snow load, and R_d and R_o are respectively the force modification factors related to ductility and overstrength and are equal to 4.0 and 1.2 for BRBFs. The structures were assumed to be located on a class C soil in Vancouver, British Columbia and to be of the normal importance category with I_E = 1.0. Dynamic analysis was performed to take advantage of the longer period T_a as allowed by the NBCC 2010. For the structures studied herein, T_{emp} = 0.025 h_n where h_n is the building height and 2 T_{emp} = 0.83 s. In all cases, the computed periods exceeded 2 T_{emp} and S(0.83 s) was therefore used in design. In Eq. (1), M_V = 1.0 for T_a less than 1.0 s.

All analyses were carried out using the ETABS 2015 computer program (CSI 2015) with translational masses in both orthogonal directions and the polar mass moments of inertia specified at every level. All members of the structures were modelled as elastic frame elements and all connections and column bases were considered as pinned. 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.

In NBCC, the ESP and RSA^{CM}-05 methods would not be permitted for the torsionally irregular buildings with B > 1.7; however, these methods were still considered here for B > 1.7 to verify the appropriateness of the code provisions. As permitted for torsionally regular buildings (B ≤ 1.7) designed with dynamic analyses (RSA), the base shear was scaled using 0.8 V instead of V as V_{RSA} was less than 0.8 V for all structures examined in this study. P-delta effects were ignored in design as they were less than 10% of the seismic loads. Storey drifts obtained from elastic analysis were multiplied by $R_d R_o/I_E$ and compared to the limit of 2.5% h_s, where h_s is the storey height, as specified for buildings of the normal importance category. For some buildings, storey drifts exceeded the code limit, but these structures were not modified to meet the drift limit.

The BRB members were designed first by assuming factored axial resistances in tension and compression equal to $\phi A_{sc} F_{ysc}$, where ϕ = 0.9, A_{sc} is the cross-sectional area of the brace core and F_{ysc} is the core yield strength which was taken equal to 280 MPa. Columns were then designed for gravity loads effects plus the seismic loads required to reach the probable axial resistance of the BRB members $T_{max} = \omega P_{ysc}$ and $C_{max} = \beta\omega P_{ysc}$, where $P_{ysc} = A_{sc} F_{ysc}$. The factor corresponding to the strain hardening adjustment factor, ω, and the compression adjustment factor that accounts for friction and Poisson's effects, β, were taken equal to 1.55 and 1.05, as obtained from tests on BRB members with capacities similar to those considered herein (Romero et al. [9]).

Ground motions

The structures were subjected to a suite of 11 pairs of orthogonal horizontal ground acceleration components. All records were obtained from the PEER Strong Motion Database [10]. The time histories were recorded at class C sites 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. For all record pairs, the first component was 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 was applied such that the 84th percentile of the geometric mean spectral ordinates did 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, and an upper bound equal to 2.0 $T_{1\text{-N-S}}$, but not less than 1.5 s. A single ground motion ensemble was used for all buildings and the lower and upper bounds of the scaling period range, 0.32 s and 2.38 s, were determined using the shortest $T_{2,\text{N-S}}$ and longest $T_{1\text{-N-S}}$ values, respectively.

INELASTIC MODELING FOR NONLINEAR ANALYSES

Solution of the dynamic equilibrium

The first strategy to solve the nonlinear equations of dynamic equilibrium in NRHA is to use the Newmark-Beta (NB) average acceleration procedure with Newton-Raphson iterations. Because material nonlinearities occur only in a small number of BRB nonlinear elements, it is also possible to reduce significantly the computational time by using the so called "*Fast Nonlinear Analysis*" or FNA as described in Wilson [11] and implemented in ETABS 2015 (version 15.0.0). As for the Newmark-Beta strategy, Rayleigh damping was used by specifying 3% of the critical value in the first mode and in the highest mode allowing an effective modal mass participation of 90%. Using a time step of 0.005 s for the same pair of ground motions, the FNA solution time was more than 2000 times faster than the classical NB solution. A comparison of the first storey drift time history response computed using the Newmark-Beta and the FNA solution strategies for BRB is shown in Fig. 2. The results are nearly identical for cases with and without consideration of P-delta effects, thus confirming the validity of the FNA solution strategy, including prediction of structural collapse due to second-order effects.

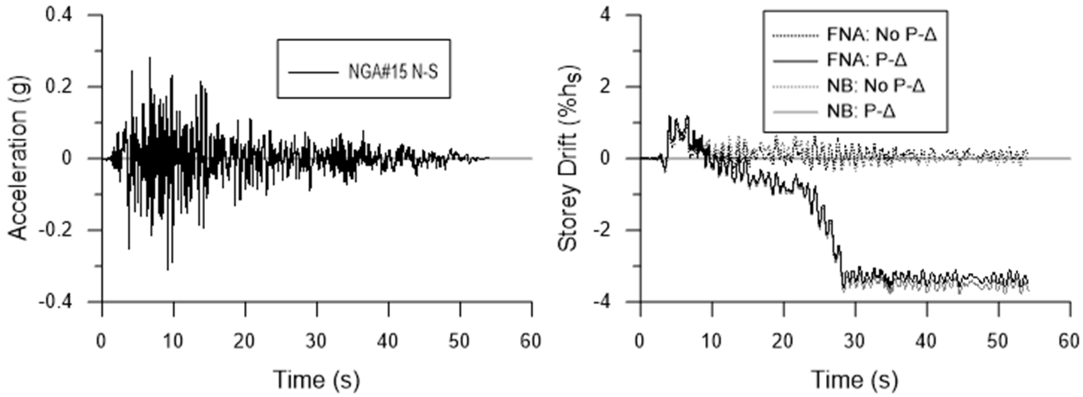

Figure 2. Comparison of the first-storey drift time history response obtained with the FNA and NB analysis methods (results shown for building B = 2.6, NBCC 2010, RSA^{CM}-05)

BRB hysteretic modelling

The nonlinear response of BRB's members is characterized by several effects such as the Bauschinger effect, kinematic hardening and isotropic hardening. On a typical backbone curve, those effects represent the smooth transition between elastic and plastic responses, the post-yield stiffness and the increase in yield load as plastic deformation occurs respectively. Due to its complexity, isotropic hardening has often been neglected in past studies on BRBFs. Also, depending on the software and the integration technique used, it is not always possible to consider isotropic hardening. The adequacy and performance of several BRB models were evaluated by Bussières et al. [12] to make recommendations for their use in research and design applications. The hysteretic responses obtained for all models compared are plotted in Fig. 3 when subjected to the loading protocol for BRB members specified in AISC 341-10 [13].

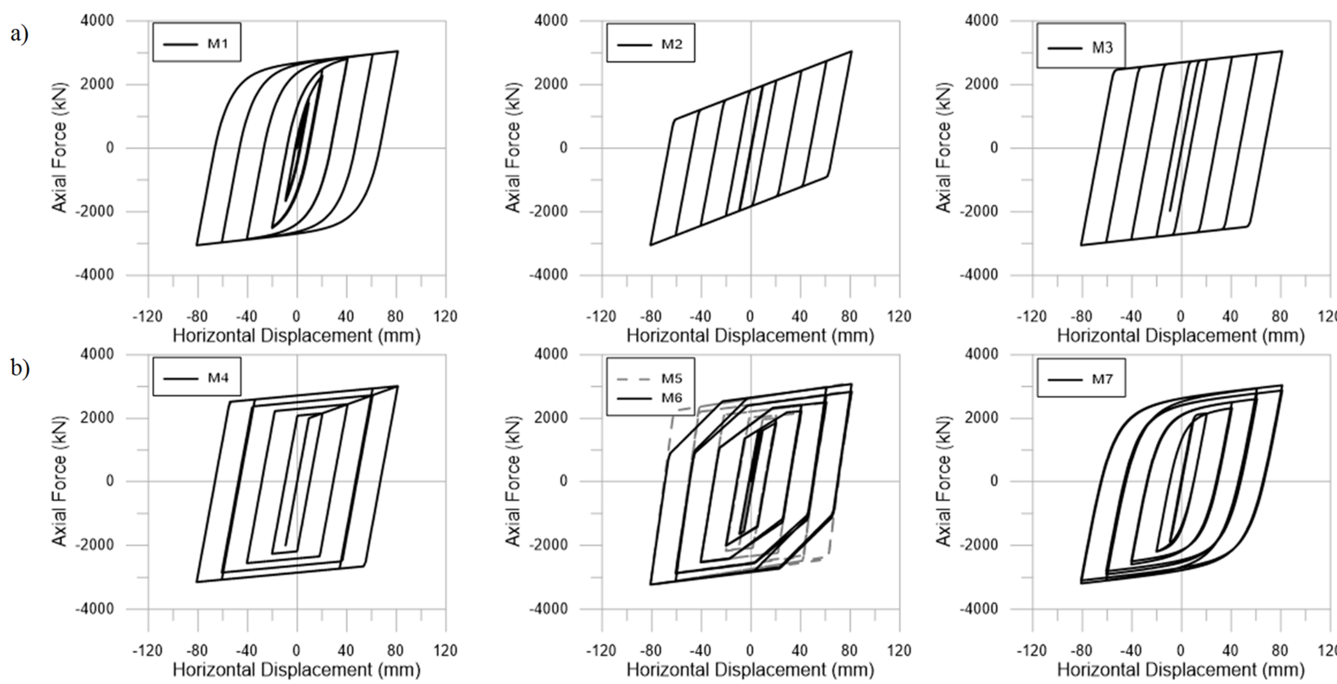

Figure 3. Hysteresis loops (BRB models) when subjected to the AISC loading protocol for: a) Bouc-Wen models; b) Nonlinear hinge, BRB hardening and OpenSees Steel02 models

Figure 3a shows three simple Bouc-Wen general plasticity models in ETABS capable of representing kinematic hardening response only. These have been widely used in past studies due to their simplicity. Figure 3b shows four more refined models that can represent both isotropic and kinematic hardening responses. Model M4 was developed by Roy et al. [7] by assigning ductile axial "frame hinges" to the BRB frame members and adding elastic "link" members acting in parallel to the brace elements. This compels the user to perform the NRHA with the lengthy NB integration scheme. Models M5 & M6 were developed by Bussières et al. [12] with the BRB hardening hysteresis parameters for nonlinear link elements. These models were introduced in ETABS 2015 and can be used with the FNA method. Model M7 was developed using the Giuffré-Menegotto-Pinto Model (Steel02) material available in the Open System for Earthquake Engineering Simulation (OpenSees) as a reference model. Further information on model parameters can be found in Roy et al. [7] for model M4 and Bussières et al. [12] for all other models.

Among the models that can be used in ETABS, the comparison with model M7 (OpenSees) showed that model M4 is the most appropriate in terms of predicting the peak storey drifts, closely followed by model M3. For BRB hardening models (M5 & M6), the stiffness is gradually increased under cyclic loading, a behaviour that is not observed in actual BRB inelastic response. These models must therefore be used with caution as this may affect the structural response. This investigation on BRB modelling suggests that model M3 is the most appropriate model to predict BRBF response, even if it does not consider isotropic hardening, if one wants to take advantage of the efficiency of the FNA technique. Model M3 was therefore selected to analyse the 35 buildings studied herein.

SEISMIC TORSIONAL RESPONSE ANALYSES

Linear response history analyses (LRHA) and NRHA were performed to compute the peak storey drifts for the 35 buildings studied. All analyses were carried out by shifting the CM of each floor diaphragm by 0.05 D_n in both orthogonal directions (RHA-05), as this was found to be the most severe loading scenario by Roy et al. [7]. In Fig. 4, the 84th percentile values of the peak N-S storey drifts obtained from RHA-05 are compared to those predicted at the design stage. The results are presented for all five levels of torsional sensitivity and for the seven different analysis methods used to account for ADE. The **r**ectangular building with B = 2.8 is referred to as "2.8**R**". Results for the methods specified in the NBCC are given in Fig. 4a. Comparisons for methods prescribed in ASCE 7, EC8, and NCh433 provisions are presented in Fig. 4b.

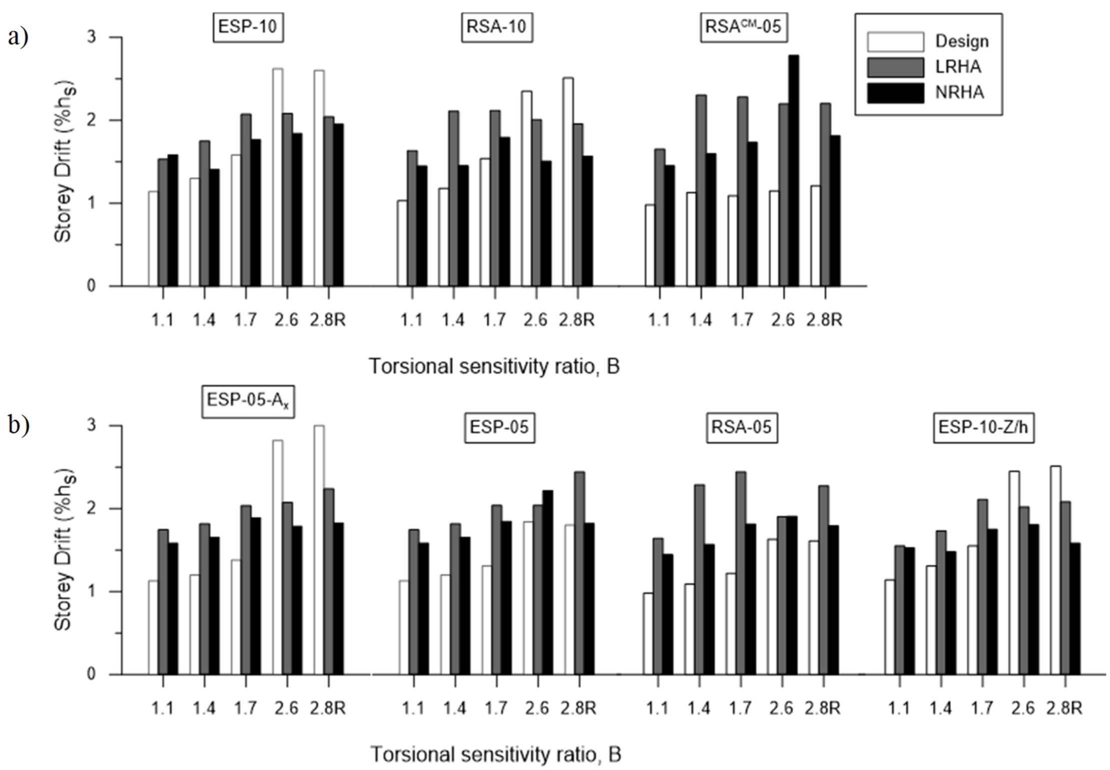

Figure 4. Design storey drifts and 84th percentile peak storey drifts from LRHA and NRHA for buildings designed in accordance with: a) NBCC; b) ASCE 7, EC8, and NCh433 codes.

Storey drifts from design and RHA both generally progressively increase with the torsional sensitivity. For most structures, NRHA gives less drift demands compared to LRHA, indicating that BRBFs possess sufficient energy dissipation capacity to control inelastic displacements and the equal displacement principle can provide conservative displacement demand estimates. When examining NRHA results for NBCC analysis methods in Fig. 4a, ESP-10 and RSA-10 gives similar displacements for B up to 1.7. For B > 1.7, the structures designed with ESP-10 experienced larger displacements, but the storey drifts are still acceptable and much lower than predicted at the design stage, indicating that the code limit for the use of the ESP-10 method could possibly be relaxed. The RSA-10 method resulted in more uniform displacements in nonlinear analyses over the entire range of buildings considered in that study, which can be attributed to the fact that RSA results were scaled to 100% V for structures with B > 1.7. For torsionally sensitive structures, the RSA^{CM}-05 method is clearly inadequate as it is unaffected by torsional sensitivity. For the B = 2.6 building, this method led to excessive storey drifts and structural collapses, confirming current NBCC limitation for its use. In Fig. 4b, storey drifts predicted at the design stage are consistently lower than NRHA values for regular buildings in torsion. The similar displacements between all methods for B ≤ 1.7 suggest that ESP-05 would be sufficient without amplification factor up to B = 1.7. Conversely, for buildings with B > 1.7, analysis methods with static torsional moments calculated with ±0.10 D_n and ±0.05A_x D_n eccentricities overestimated peak displacements compared to nonlinear seismic demand. The RSA-05 is the most accurate to predict peak storey drifts for torsionally irregular buildings (B > 1.7). This indicates that the RSA-10 method in NBCC could possibly be relaxed to RSA-05 for torsionally regular and irregular structures. The ESP-10-Z/h method specified in Chile gives equal or lower displacements compared to the ESP-10 method, even for B = 2.6 or 2.8R. This suggests that reducing design torsional moments along the building height in design may not be detrimental to torsional response and could lead to a better distribution of the inelastic demand along the building height.

A clear demarcation was also observed between two groups of torsional sensivity: for B less than 1.4, drifts were generally slightly larger at the eastern end, namley the side towards which the CM was displaced in the analysis; for B ≥ 1.4, peak displacements were always observed at the edge opposite to the eccentricity. This behaviour is illustrated in Fig. 5 for the square buildings with B = 1.1 and 2.6 under NGA#15 ground motion. In Fig. 5a, the largest storey drift is observed at the East end for B = 1.1 and at the West end for B = 2.6. As shown in Fig. 5b, for the latter, the torsional response is much more pronounced and progressively increases during the earthquake, which resulted in peak storey drift along the East wall occuring in the opposite direction. This response is counterintuitive but this and past researchs [6][7] indicate that it reflects the actual behaviour of torsionally sensitive structures. Except for the low B structures, inaccuracies in storey drift predictions mainly result from actual torsion response not corresponding to that assumed in design. Hence, although in-plane torsional moments included in design helped controlling the structure torsional response, correlation between predicted and NRHA torsionally induced displacements was essentially coincidental in view of the differences between assumed and actual torsional responses. For torsionally sensitive structures, new design analysis methods are needed to properly account for ADE effects.

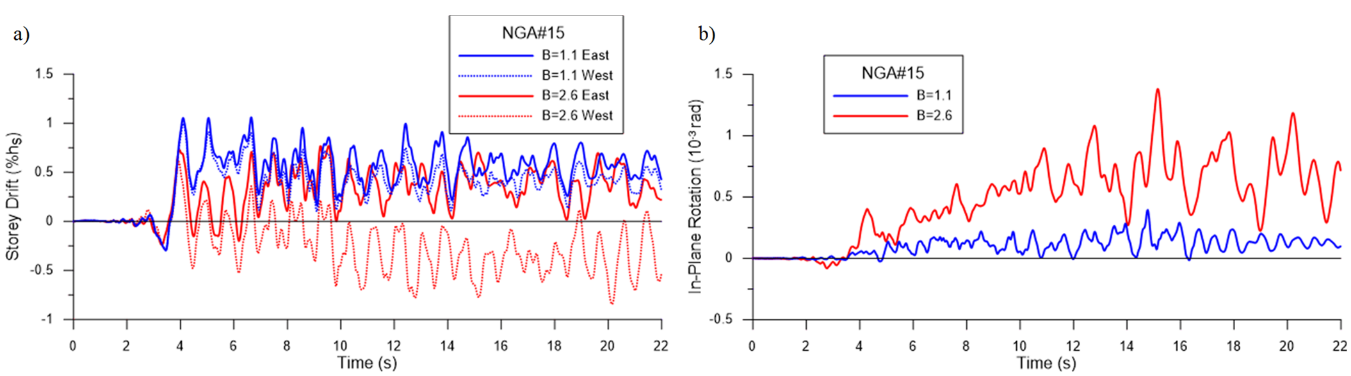

Figure 5. Time history of the torsional response at the first-storey of the square buildings with B = 1.1 and B = 2.6.

CONCLUSIONS

For the BRBFs studied, the computational time for NRHA using the FNA method in ETABS was up to 2000 times shorter than that required for the classical Newmark-Beta average acceleration procedure. Both methods were found to give the same results, including P-delta effects, when using the same BRB nonlinear models. Among the nonlinear models available in ETABS that can be utilized with FNA, properly calibrated bilinear Bouc-Wen model M3 that accounted for kinematic strain hardening was found to be the most appropriate as it gave acceptable results when compared to more refined models that included both isotropic and kinematic hardening responses.

For structures with B ≥ 1.4, the maximum storey drifts occurred at the building edge located on the opposite side of the accidental eccentricity considered in NRHA. This is contrary to the behaviour assumed in design. The Equivalent Static Procedure with static torques due to ±0.10 D_n eccentricity in NBCC was found appropriate for all buildings studied. The same method with torsional moments reducing along the structure height, as prescribed in the Chilean code, gave comparable results and would also be adequate for all B values considered herein. Regardless of the method used to account for ADE, the maximum storey drifts predicted in design underestimated NRHA values for regular buildings in torsion (B ≤ 1.7). For B > 1.7, the NBCC ESP-10, NBCC RSA-10, ASCE ESP-05-A_x, and NCh433 ESP-10-Z/h analysis methods overestimated storey drifts. The EC8 RSA-05 method was the most accurate to predict peak storey drifts for irregular buildings in torsion (B > 1.7). For B up to 1.7, RSA and ESP analysis methods could potentially be relaxed by considering only ±0.05 D_n eccentricity, as permitted in EC8. The results also showed that the RSA-05 can be used for buildings with B larger than 1.7. However, the NBCC RSA^{CM}-05 method should not be used for these structures, as is currently the case in NBCC 2010.

The study confirmed the necessity of considering accidental eccentricity in the seismic design of symmetrical structures. It also showed that current code analysis methods need to be revised to achieve more consistent performance across the range of torsional sensitivity levels, in particular for buildings with B > 1.7. This study was limited to structures with symmetrical SFRS and mass arrangements and future studies should examine buildings with unsymmetrical SFRS and mass properties. Research should also be conducted on taller buildings with seismic response more significantly influenced by higher vibration modes.

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