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Towards the Quantitative Performance Evaluation of NBC Design Requirements for Torsionally Sensitive Buildings

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ABSTRACT

The National Building Code of Canada (NBC) considers the torsional effects in structures based on a torsional sensitivity parameter, B, defined as the ratio of the maximum lateral storey displacement to the average of the storey displacement. If this parameter exceeds 1.7, NBC categorizes the structure as a torsionally sensitive structure. Unless for certain seismic categories, a dynamic analysis procedure needs to be carried out for torsionally sensitive structures, and the equivalent static force (ESF) procedure is not permitted. In the US, FEMA P-2012 has shown that the torsional design requirements of the ASCE/SEI 7-16 design code are generally conservative for most building configurations and has proposed recommendations to relax some of these requirements. A similar study needs to be conducted in Canada to evaluate the effectiveness of torsional design requirements of NBC on the seismic performance of structures. This paper presents an initial phase of a comprehensive project that is being carried out to assess the seismic performance of buildings in Canada with different configurations of seismic force resisting systems. As part of this project, two single-storey nonlinear three-dimensional models are created in OpenSees to represent the aggregate behaviour of two regular steel moment-resisting frame buildings with different fundamental periods located in Montreal. The configuration of moment-resisting frames of the two reference models is then changed to simulate the behaviour of torsionally sensitive buildings as specified in NBC. All four models are designed such that they meet the drift and strength requirements of NBC. The capacity of the models, determined by incremental dynamic analyses, are compared and used to assess the effectiveness of torsional design requirements of NBC for irregular buildings. The study found that the collapse capacity of drift-controlled buildings with torsional irregularities can be considerably higher than regular buildings which is consistent with the findings presented in FEMA P-2012.

Keywords: Torsionally sensitive buildings, irregular structures, collapse capacity, incremental dynamic analysis, nonlinear modeling

INTRODUCTION

Asymmetric (or irregular) plan buildings experience coupled translational and torsional motion, when excited laterally. Torsional effects may significantly modify the seismic response of buildings, and they have caused severe damage or collapse of structures in several past earthquakes [1]. These effects occur due to different reasons, such as plan mass eccentricity, uneven stiffness and strength distribution among seismic force resisting elements, torsional components of the ground movement, etc. Seismic force resisting elements in buildings with significant torsional effects could experience large displacements and are more prone to damage due to the non-uniform demand on the elements.

The significance of structural irregularity and torsion is to an extent where building design codes have separate requirements and seismic provisions for irregular structures. In ASCE 7-22 [2], for instance, horizontal torsional irregularity and extreme horizontal torsional irregularity are defined to exist if the maximum story drift is more than 1.2 and 1.4 times the average story drift, respectively. These values are computed including accidental torsion, which can be accounted for by applying the seismic force with 5% eccentricity with respect to the mass center in the direction perpendicular to the direction of earthquake. ASCE 7-22 has specific requirements for the design of buildings that have horizontal torsional irregularity. The accidental torsional moment at each story level of these buildings must be multiplied by a torsional amplification factor (A_x), as illustrated in Figure 1. Also, the redundancy factor of torsionally irregular structures must be taken as 1.3 which results in 30% increase in the design base shear compared to regular structures. The code also requires checking the storey drift at the edges of irregular buildings (where the drift is the largest) instead of using an average value or the drift at the mass center. There are other requirements such as amplification of the design force for collectors, use of orthogonal load combinations and limiting applicability of the Equivalent Static Force (ESF) procedure based on the seismic design category of the structure.

In the National Building Code of Canada (NBC) [3], torsional effects in structures are assessed based on a torsional sensitivity parameter, B, which similar to ASCE 7-22, is defined as the ratio of the maximum and average story drifts, as shown in Figure 1. If B exceeds 1.7, the structure is categorized as a torsionally sensitive structure which for most seismic categories, a dynamic analysis procedure needs to be carried out and the ESF procedure is not permitted. In the dynamic analysis procedure, a lateral earthquake force that is at least 100% of the base shear determined by the ESF procedure should be used. However, for structures that are torsionally regular ($B \leq 1.7$), the NBC permits a reduction of up to 80% of the ESF procedure's base shear for the lateral earthquake force if dynamic analysis is performed. To consider the effect of accidental torsion on torsionally sensitive buildings, the NBC requires applying torsional moments of $\pm 0.10 D_{nx} F_x$ at each floor, where F_x is the seismic force at floor *x* determined from elastic dynamic analysis and D_{nx} is the dimension of floor *x* perpendicular to the earthquake direction. For torsionally regular buildings, the NBC allows using a three-dimensional dynamic analysis and shifting the center of mass by a distance of $\pm 0.05 D_{nx}$ to account for the accidental torsion effect. This results in a torsional moment that is approximately equal to half of that considered for the torsionally irregular buildings. Furthermore, similar to the ASCE 7, NBC requires checking the deflection limit at the point that has the maximum storey drift on the floor plan. Since irregular buildings typically have significantly larger deflections at the edges, meeting the storey drift limit for these structures can be challenging and may require considerable increase in the structural stiffness and strength.

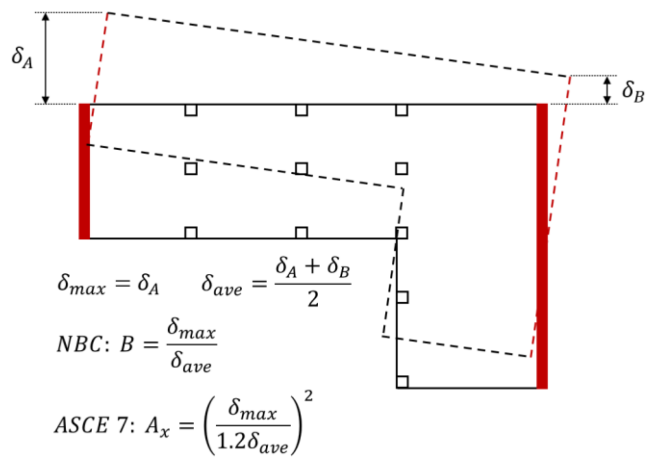


Figure 1. Torsional sensitivity factor (B) and Torsional amplification factor (A_x) used in NBC [3] and ASCE 7 [2]

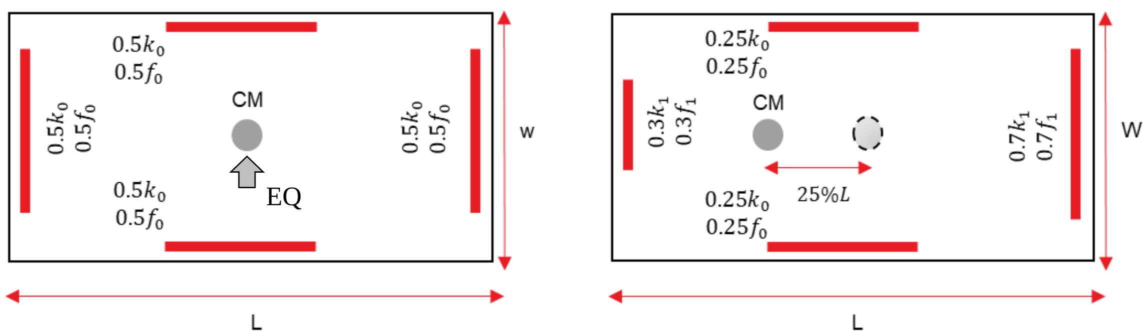
In FEMA P-2012 [1], a comprehensive study has been done on buildings with torsional irregularities to determine the effectiveness of ASCE 7-16 [4] design requirements for irregular structures and propose modifications if needed. Over 2,000 single-story, nonlinear three-dimensional archetype models with varying degrees of torsional irregularity calibrated to represent the behaviour of multi-story buildings were used in the study. By changing the location of lines of resistance, archetypes with different degrees of torsional irregularity were created. The study concluded that the ASCE 7-16 design requirements for torsionally irregular buildings tend to be overly conservative for the majority of building configurations. However, it also noted that the accidental torsion amplification and satisfying drift and stability limits are crucial design requirements to ensure adequate resistance against collapse in torsionally irregular buildings. The study provided several recommendations to avoid unnecessary conservatism in the design of torsionally irregular structures including relaxing the amplification of seismic force due to the redundancy factor and allowing the use of ESF procedure for torsionally irregular buildings.

There has not been a similar study in Canada, even though the design requirements for torsionally sensitive structures in NBC are different than those provided in ASCE 7. Recently, the National Research Council Canada (NRC) in collaboration with Carleton University have initiated a comprehensive research project to systematically evaluate the seismic performance of buildings with different configurations of seismic force resisting systems (SFRSs). This paper provides a summary of a preliminary study that was carried out as part of this project to compare the performance of regular and torsionally sensitive structures designed based on the NBC requirements. The study includes conducting pushover and incremental dynamic analyses (IDA) on single-storey nonlinear three-dimensional models of two archetypes with different fundamental periods which were designed for the same base shear twice: first assuming a symmetrical plan and second assuming a torsionally irregular plan for the building. By comparing collapse capacities obtained from the IDA analyses, the seismic performance of regular and torsionally sensitive buildings was evaluated. More analyses are currently underway to verify the preliminary results and extend the study to other SFRSs configurations and multi-storey nonlinear models that can provide a more realistic representation of the structural behaviour.

ARCHETYPE DESIGN AND MODELLING

First, two baseline archetypes were designed to represent the aggregate behaviour of two steel moment-resisting frame buildings located in Montreal. One archetype had a fundamental period of 1.0 s (will be referred to as Case 1), while the other one was significantly stiffer and had a fundamental period of 0.26 s (will be referred to as Case 2). As shown in Figure 2(a), the two archetypes had an identical symmetrical plan (63 m long by 45 m wide) with two lines of resistance in each direction. To simplify the design process, the seismic weight of the archetypes was kept the same (60,000 kN) and only the lateral stiffness was changed to investigate the effect of natural period on the seismic performance.

The two baseline archetypes had a symmetrical plan and therefore represented the behaviour of regular buildings ($B \leq 1.7$). By changing the configuration of the resisting elements, two additional archetypes were designed to simulate the behavior of torsionally sensitive buildings ($B > 1.7$) as specified in NBC. These archetypes will be referred to as Case 1-T and Case 2-T from hereon. To increase the torsional sensitivity factor, the following modifications were made to the resisting elements of the baseline archetypes: 1) uneven distribution of the stiffness and strength of resisting elements in the loading direction, 2) reduction of the strength and stiffness of the orthogonal resisting elements, and 3) introducing 25% mass eccentricity to the structure. The plan view of Case 1-T and Case 2-T archetypes is shown in Figure 2(b).



(a) *(b)*

Figure 2. Plan view of: (a) baseline archetypes (Case 1 and Case 2) and (b) torsionally sensitive archetypes (Case 1-T and Case 2-T)

Archetype buildings were represented by single-story, nonlinear three-dimensional models developed in the OpenSees analysis software, as shown in Figure 3. One-storey models have been commonly used in the literature to represent the aggregate behavior of multi-story buildings and investigate the effects of torsional irregularities in the building plan [1, 5]. These models are highly efficient, making them suitable for conducting parametric studies and collapse assessment using incremental dynamic analysis. FEMA P2012 [1] compared the response of a series of multi-storey models with their equivalent one-storey models and demonstrated that one-storey models are capable to represent the nonlinear torsional behaviour of buildings with symmetric and asymmetric plans with reasonable accuracy.

Each lateral force resisting element of the archetype structures was represented as a moment frame in the equivalent one-story model. The properties of the moment frames were defined such that they only contributed to the stiffness and strength of the structure in the frame direction and had near-zero rigidity in the direction perpendicular to the frame. With this approach, each moment frame basically acted as a uniaxial nonlinear spring which simplified evaluation of the structural behaviour in the direction parallel and perpendicular to earthquake. Each moment frame consisted of two columns which were fixed at the base and connected to each other at the top using a stiff beam. The beam and columns were modelled using the *nonlinearBeamColumn* elements of OpenSees. The beam was modelled with a linear elastic material, while the column was represented with the Ibarra-Medina-Krawinkler (IMK) nonlinear cyclic model [6]. The backbone curve of the IMK model was defined according to the strength and stiffness values assigned to the resisting elements of the archetypes. This required calibrating the moment-curvature response of columns so that their lateral stiffness matches the stiffness of resisting elements. The total lateral stiffness and strength of the archetypes were determined such that they meet the drift and strength requirements of NBC 2020 [3]. A relatively low cyclic deterioration was considered for the IMK model since previous studies have shown that the influence of cyclic deterioration on the collapse capacity of structures is insignificant [6]. The in-plane degrees-of-freedom of all the nodes at the first storey were constrained assuming the floor is stiff enough to provide a rigid diaphragm for transferring lateral loads. The mass center was defined by adding an additional node to the model at the centroid of the plan in the first storey.

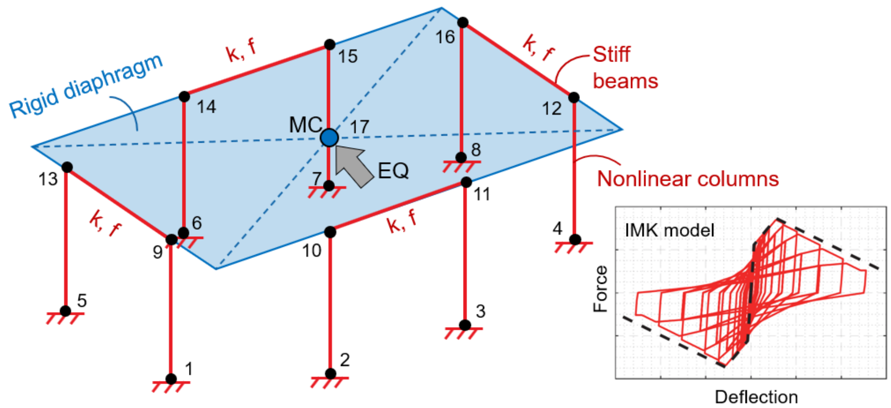


Figure 3. Equivalent one-storey three-dimensional model of the reference archetype

Using the analytical model, each archetype was designed to meet the strength and drift requirements of NBC 2020. The design procedure comprised of the following steps:

1. Determine the design spectral acceleration corresponding to the fundamental period of the structure estimated from mass and stiffness in the loading direction.
2. Determine the specified lateral earthquake force (V_s) using the ESF procedure of NBC.
3. Determine the torsional sensitivity factor (B) by applying the design seismic force at the distance ±0.10L from the mass center. If $B \leq 1.7$, reduce the design seismic force to $0.8V_s$. This is based on the assumption that the design force from the dynamic analysis will be less than or equal to 80% of V_s obtained from the ESF method.
4. Set the total yield strength of the structure equal to the design seismic force determined from Step 3. The ultimate strength of the structure is taken as 1.1 times the yield strength. Define a trilinear backbone curve for each resisting element based on the stiffness and strength factors shown in Figure 2.
5. Check if the maximum drift of the structure computed under the design seismic force meets the drift limit of $0.025h_s$ specified in NBC. If the drift limit is not satisfied, increase the stiffness and strength of resisting elements, and repeat the design steps until the structure meets the drift limit.

Table 1 summarizes the structural characteristics of the four archetypes after the design was completed. It can be seen that Case 1-T had considerably higher stiffness and strength values than Case 1. This is because the design of Case 1-T was controlled by deflection which means that the lateral stiffness had to be substantially increased in order for the structure to meet the inter-storey drift requirement. Increasing the stiffness reduced the fundamental period of the structure resulting in a higher design seismic force which required increasing the strength of the structure. The higher design seismic force also increased the lateral deflection which required increasing the lateral stiffness even more. On the other hand, the deflections of Case 2 and Case 2-T were relatively small and did not control the design. Thus, no significant increase in stiffness and strength was needed for the design of Case 2-T compared to Case 2.

Table 1. Design characteristics of archetypes

Case study	Torsional sensitivity factor, B	Total stiffness, K ($\times10^3$ kN/m)	Fundamental Period (s)	Design spectral acceleration (g)	Specified seismic force, V_s (kN)	Total yield strength F_y (kN)	Total ultimate strength, F_u (kN)	Maximum disp., D_{max} (mm)
Case 1	1.13	235	1.00	0.26	2120	1694	1863	61.0
Case 1-T	1.82	1029	0.48	0.49	3936	3936	4330	78.0
Case 2	1.13	3529	0.26	0.65	5216	4173	4590	10.3
Case 2-T	1.85	4411	0.23	0.77	6176	6176	6794	29.5

COLLAPSE ASSESSMENT

First, a pushover analysis was conducted on each model to evaluate the distribution of lateral force between the elements of SFRS and to ensure that the model is performing as intended. In addition to the pushover analysis, an incremental dynamic analysis (IDA) [7] was carried out to assess the seismic performance and collapse capacity of the structure. With the IDA method, the structure is subjected to a set of ground motion records each scaled from low to high seismic intensities such that the entire range of structural response, from elastic to inelastic and ultimately collapse, can be simulated. In this study, the selection of the ground motion set and the evaluation of collapse capacity were conducted according to the FEMA P-695 [8] procedure. Three criteria were used to determine the spectral acceleration corresponding to the collapse point: 1) drift ratio exceeding 10% at any location in plan, 2) flattening of the IDA curve (the last point on the IDA curve with the tangent slope equal to 20% of the elastic slope), and 3) non-convergence in the nonlinear solution. The first criterion was chosen based on the recommendation of FEMA P-2012 [1] and the next two criteria were selected according to Vamvatsikos and Cornell [7]. Using collapse capacities obtained from all ground motion records, the fragility curve of the archetype which indicates the probability of collapse for different levels of spectral acceleration was generated.

Figure 4 compares the nonlinear pushover responses of the four archetypes. It can be seen that the torsionally sensitive archetypes (Case 1-T and Case 2-T) could only reach to about half of their full design ultimate strength presented in Table 1. Because of the torsional behaviour, the frame closer to the mass center experienced much larger deformations than the other frame and as a result the two frames could not develop their ultimate strength at the same time (when the left frame reached its ultimate strength, the right frame was still at the elastic portion of the response). In spite of this, Case 1-T was able to provide a stronger pushover response than its baseline archetype, Case 1. This is because, as discussed in the previous section, Case 1-T was a drift-controlled archetype requiring substantial increase in stiffness and strength to limit deflections due to torsional movement. However, this was not the case for the Case 2-T archetype where the design was controlled by strength rather than the inter-storey drift. As shown in Figure 4, the pushover response of this archetype was considerably weaker than its baseline case (Case 2).

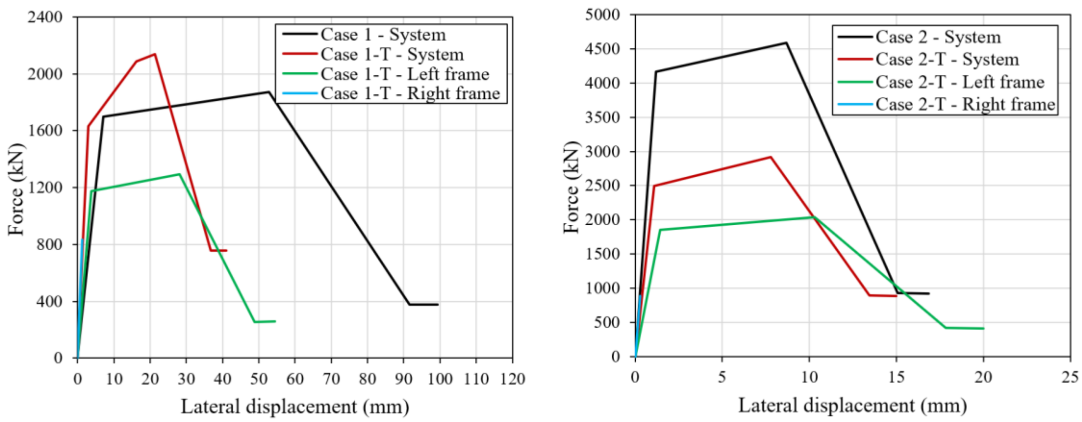


Figure 4. Comparison of pushover responses of symmetrical and torsionally sensitive archetypes

Figure 5 shows sample IDA curves calculated for the four case studies. Figure 6 also compares the fragility curves generated based on the collapse points determined according to the previously mentioned criteria. It can be seen from Figure 6 that Case 1-T has a higher collapse capacity compared to Case 1 which is expected since Case 1-T also has a stronger pushover response. The mean collapse capacities of Case 1-T and Case 1 are 0.55 g and 0.32 g, respectively. Interestingly, the mean collapse capacity of Case 2-T is also slightly higher than its baseline archetype (0.54 g versus 0.50 g) despite the fact that Case 2-T had a considerably weaker pushover response than Case 2. The higher collapse capacity of Case 2-T can be attributed to its higher mass moment of inertia resulted from the large mass eccentricity required to increase the torsional sensitivity of the archetype. As the mass moment of inertia increases, the resistance of the structure against rotation increases, which can lead to a better seismic response.

The analysis results of this preliminary study are consistent with the findings of FEMA P-2012 which demonstrated that the seismic performance of torsionally irregular buildings that meet design code requirements is similar or in some cases even better than that for regular buildings with similar characteristics. In particular, FEMA P-2012 found that the collapse capacity of drift-controlled buildings with torsional irregularities can be substantially higher than regular buildings since satisfying the drift limit for torsional irregular buildings can result in significant overstrength in the system. Similar conclusion was made in the current study. The preliminary results of this study demonstrate that there is a need for a systematic investigation to gain more insight into the seismic behaviour of torsionally sensitive buildings and to evaluate the effectiveness of NBC requirements for design of these structures.

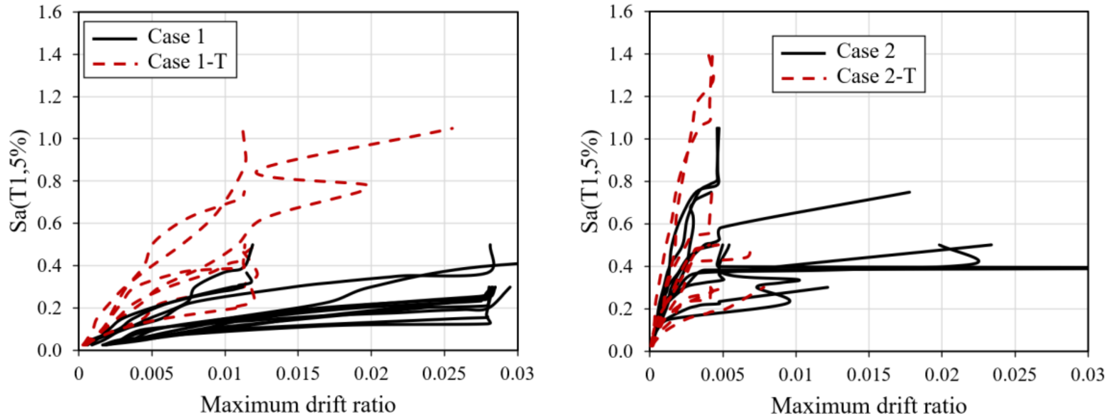


Figure 5. Comparison of sample IDA curves of symmetrical and torsionally sensitive archetypes

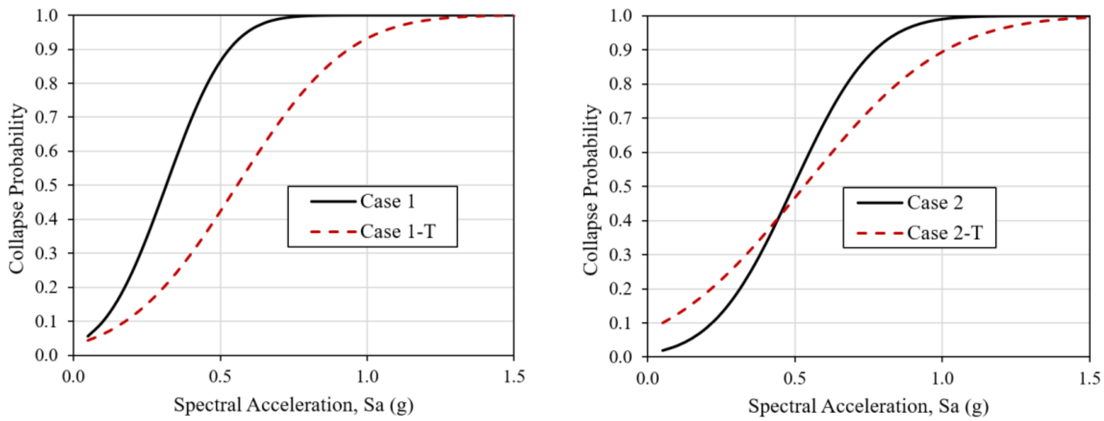


Figure 6. Comparison of fragility curves of symmetrical and torsionally sensitive archetypes

SUMMARY AND CONCLUSIONS

This paper presented the preliminary results of a collaborative study between Carleton University and NRC that is currently underway to evaluate the seismic performance of torsionally sensitive buildings in NBC. Two single-storey nonlinear three-dimensional models were created in OpenSees to represent the aggregate behaviour of two regular steel moment-resisting frame buildings with different fundamental periods located in Montreal. The configuration of the moment-resisting frames of the two reference models was then changed to simulate the behaviour of torsionally sensitive buildings. All four models were designed such that they met the drift and strength requirements of NBC. The capacity of the models, determined by incremental dynamic analyses, were compared and used to assess the effectiveness of design requirements of NBC for irregular buildings.

The analysis results showed that the seismic performance of torsionally sensitive buildings designed according to the NBC requirements is satisfactory and, in some cases, can be even better than that for regular buildings with similar characteristics. For long-period (mid- and high-rise) buildings with torsional irregularities, satisfying the drift limit under a combination of translational and rotational movement can be challenging and may require significant increase in the structural stiffness. Since the stiffness and strength of a structure are related to each other, increasing the stiffness results in higher overstrength and better seismic performance. The conclusions made from this study are consistent with the findings of FEMA P-2012 on the seismic performance of torsionally irregular buildings designed according to the ASCE 7. However, the findings presented here are based on a limited number of analyses and archetype configurations and cannot be used to draw general conclusions. A comprehensive study is currently underway to provide a better understanding of the seismic performance of torsionally sensitive buildings by conducting more systematic analyses and investigating other types and configurations of SFRSs.

ACKNOWLEDGMENTS

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