

Seismic-Response Analysis of RC C-Shaped Core Walls Subjected to Combined Flexure, Shear, and Torsion

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Abstract: Although dynamic torsional responses are often regarded as secondary effects in the seismic design of symmetric-plan buildings, torsional effects arising from plan asymmetry can be substantial in some cases. In RC wall buildings, a combination of flexural, shear, and torsion dictates the type of failure, which needs to be addressed in the design of RC walls. This study investigates two aspects of building structures with C-shaped RC walls: (1) evaluation of seismic force demand at different levels of torsional sensitivity, and (2) effectiveness of using the dual plastic hinge method in controlling the seismic shear force demand. A macroscale modeling approach using the wide column analogy captures the inelastic response of C-shaped RC wall buildings, including torsional effects. The numerical model of the wall is validated against available experimental data. Nonlinear time history analyses of typical multistory buildings located in eastern North America (8-, 12-, and 16-story) are performed. Four different levels of torsional sensitivity ($B = 1.3, 1.7, 2.0$, and 2.5) are considered for each building configuration. Although the studied range of torsional sensitivity has no substantial effect on the bending moment envelope of the building, it significantly increases the story shear force demand during an earthquake. The shear force envelopes in torsionally sensitive buildings ($B > 1.7$) exceed the capacity design envelope predicted by the response spectrum analysis of a large number of selected ground motions. The dual plastic hinge design method recently proposed by other researchers is adopted and is found to be an efficient method, resulting in lower shear force demand along the height of the structure, especially when high torsional flexibility is expected. DOI: 10.1061/(ASCE)ST.1943-541X.0002181. © 2018 American Society of Civil Engineers.

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Introduction

The capacity design method is the base for the seismic design of shear walls in many countries, such as Canada and New Zealand. Although current civil engineering practice prefers to use the nonlinear static procedure because of its simplicity and consistency in the design process, dynamic time history or response spectrum analyses (RSAs) are inevitable in some cases. According to the National Building Code of Canada (NBCC 2015), dynamic analysis is mandatory if the torsional sensitivity (irregularity) of the building, B , exceeds the limit of 1.7. Torsional sensitivity in a building is determined based on the maximum value B among all floors, where B_x computed at floor level x is $B_x = \delta_{\max}/\delta_{\text{ave}}$. Similar classification is specified by ASCE/SEI 07-16 (ASCE 2016). However, ASCE 07-16 includes two levels of torsional irregularities for torsionally sensitive buildings, the definition of which differs slightly from the NBCC 2015 definition. According to ASCE 07-16, torsional sensitivity is the ratio of maximum to average story drift in a story, and torsional irregularity corresponds to buildings in which this ratio exceeds 1.2 but is not more than 1.4, which is categorized as the limit for extreme torsional irregularity. The aforementioned limits are specified for nonflexible diaphragms only (ASCE 2016). Fig. 1 shows a schematic

view of asymmetry in a building plan and the torsional sensitivity according to the NBCC 2015. Although the code does not prescribe a specific type of dynamic analysis, the response spectrum analysis method is the most preferable technique to predict the seismic response of structures. For shear wall structures, this technique provides an accurate and adequate estimation of response parameters when the wall behaves within the linear range. The linear behavior, however, is not the realistic response of the wall in many cases, and fluctuation of both the flexural and shear stiffness is expected when the structure is subjected to a severe earthquake (Luu et al. 2013). The code simply applies some reduction factors to the predicted force demand from an elastic analysis to consider the response nonlinearity in the design. Because the force redistribution upon the nonlinear actions in the wall cannot be captured by the elastic modal analysis, unrealistic effects from the higher modes will be induced, and consequent inaccuracies may occur in the seismic response predictions of the RC wall (Pulay and Priestley 1992; Luu et al. 2013).

In the seismic analysis of RC wall structures, higher-mode effects can significantly amplify the force demands (bending moments and story shears) over the height of the RC wall system, potentially leading to unexpected damage or possibly failure (Wiebe and Christopoulos 2009). As a result of early New Zealand investigations (Blakeley et al. 1975), the 1982 New Zealand Design Standard NZS 3101 (NZS 1982) introduced a simple dynamic shear amplification factor to account for the higher-modes effects on the design forces of the structure. Accordingly, a new clause (21.5.2.2.7) was added in the recent version of the Canadian code CSA A23.3-14 (CSA 2014), which introduced a dynamic shear amplification factor, ω_n , to account for the inelastic effects of higher modes in RC walls, except for coupled and partially coupled wall structures. This exception can be explained in part by the fact that, unlike cantilever walls which deform entirely

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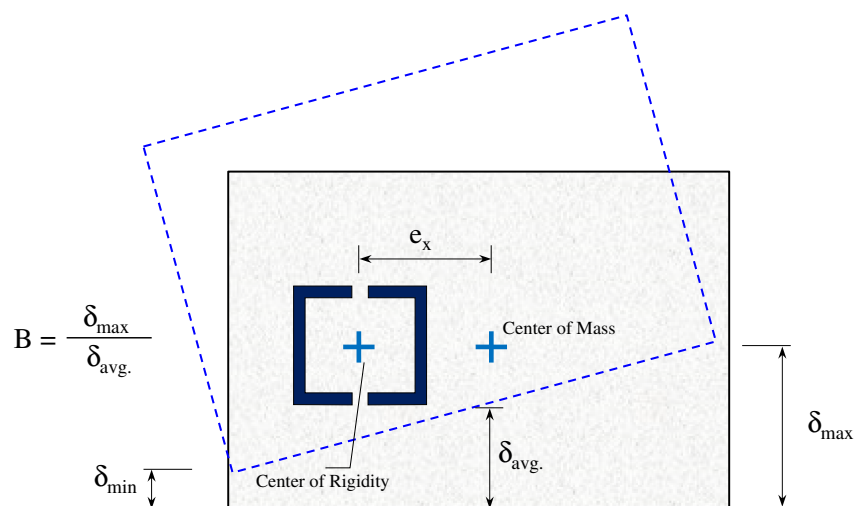


Fig. 1. Schematic view of asymmetric building plan.

in a flexure mode, the deformation of coupled walls includes a significant shear (sway) mode due to the deformation of the coupling beams. In contrast, despite several observations reported in the literature, codes in the United States have not yet considered the significant effect of higher modes on seismic demands in buildings with a cantilever RC wall system.

In seismic design of RC wall structures, story shear force demands can be of special interest as an important response parameter. As mentioned previously, the 2004 version of CSA A23.3-04 (CSA 2004) did not consider the shear increase due to inelastic effects of higher modes (Boivin and Paultre 2010). Pelletier and Léger (2017) showed that the new dynamic shear amplification factor prescribed by CSA A23.3-14 results in a more realistic seismic shear force demand than do nonlinear time history results. However, an increase in the shear force demand was reported by increasing the torsional sensitivity of the building. Because the results of the nonlinear time history analysis (NTHA) are affected by several parameters such as the building configuration and the ground motion selection, more studies need to be conducted to highlight the effect of the high torsional sensitivity of the building in its seismic response.

Realistic estimation of seismic force demand is more essential when high torsional effects in the seismic response of structure are expected. Few studies have investigated how torsional effects can affect the seismic force distribution in building structures. Fajfar et al. (2005) found that inelastic torsional effects in the seismic response of the structure depend on the ductility demand and thus on the intensity of selected ground motion. A substantial contribution from torsional effects can be expected in some cases, which generally decreases with increasing plastic deformations, and leads to significant variations in the seismic force demand of the structure. A study by Dubey and Sangamnerkar (2011) on seismic behavior of asymmetric RC buildings showed that torsional analysis of buildings should not be considered as a secondary analysis only. Structural damages during past wind storms and earthquakes reveal that torsion is the most critical factor in some cases, leading to major damage or complete collapse of buildings. Herrera et al. (2013) conducted a study subsequent to the collapse of a commercial building and a school building categorized as structures with high torsional risk and discontinuity in diaphragms during the earthquake of Cariaco, Venezuela (1997). They investigated the seismic response and torsional effects of RC structures with irregular plans and variations in diaphragms, designed with Venezuelan codes.

As expected, the observed internal forces due to torsional effects on the columns located in the core of the building were less than the corresponding force for the columns of outer axis, which confirmed the design presumption for reducing the stress concentration in those structural members.

This paper quantitatively evaluates the seismic response of C-shaped RC core walls with different torsional sensitivity factors and assesses the performance of the dynamic shear amplification factor prescribed by CSA A23.3-14 in predicting the story shear force demand of buildings with a high level of torsional sensitivity. This evaluation was done by comparing the seismic response prediction from the RSA with the results of NTHA. A three-dimensional (3D) macromodel was developed using OpenSEES version 2.4.4 (McKenna et al. 2013) based on the modified wide column model (WCM) proposed by Arabzadeh and Galal (2017) for simulating the nonlinear response of RC cores, including shear deformation nonlinearity. The model was originally proposed by Beyer et al. (2008) and was extended by Pelletier and Léger (2017). The accuracy of the modeling approach was evaluated against the available experimental test data from the literature. The validated numerical model was then used to investigate the seismic response of 8-, 12-, and 16-story RC building structures located in eastern North America using NTHA. A group of 48 artificial records proposed by Atkinson (2009), including near-distance and far-distance ground motions with different magnitudes, were used for NTHA of the RC core in different levels of torsional sensitivity. The story shear force envelopes from individual ground motions were compared afterward with the corresponding story shear resistance, V_r , resulting from the seismic design based on RSA.

The capacity design proposed for design of cantilever walls in the Canadian design code (CSA A23.3-14), as well as in EuroCode EC8 (CEN 2006) and the New Zealand seismic standard NZS-3101, (NZS 2006), is based on a plastic hinge at the base of the wall and assumes an elastic response along the rest of the wall height. Although these codes consider a linear variation for the flexural design envelope to take higher mode effects into account, Panneton et al. (2006) and Priestley et al. (2007) showed that such an assumption is subjective. In other words, this assumption does not always preclude the spread of plasticity into the upper regions, and subsequently, sufficient protection against yielding in the upper portions of the walls cannot be provided. Priestley et al. (2007) proposed a bilinear flexural design envelope to address this issue.

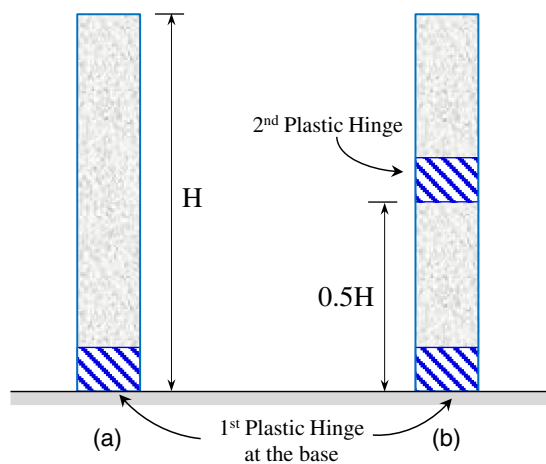


Fig. 2. General layout of SPH and DPH models for RC walls.

Based on the results of a statistical study, Wiebe and Christopoulos (2009) found that providing multiple plastic hinges along the height of the walls (acting as nonlinear fuses) would be advantageous. Hence, with no noticeable increase in the maximum displacement, the peak flexural demand can be limited, and this will prevent it from exceeding the design envelope. Considering this, Panagiotou and Restrepo (2009) proposed a dual plastic hinge (DPH) design approach useful for the design of high-rise RC structural walls. Fig. 2 presents a general layout of the DPH model for RC walls. The approach introduces a second hinge at an intermediate height of the cantilever wall, which is intended to reduce the effects of higher modes of response in buildings. Moreover, the single plastic hinge (SPH) design approach requires large amounts of longitudinal reinforcement in the intermediate portion of the walls, and this is associated with significant congestion and higher cost (Panagiotou and Restrepo 2009).

Hence, this study adopted a DPH configuration to evaluate the effectiveness of the proposed method in controlling the story shear demand in torsionally sensitive structures. The following sections compare the results of the analysis of the 8-, 12-, and 16-story buildings using SPH and DPH approaches.

Analysis Methodology

Among the various simulation techniques proposed in the literature for predicting the response of shear/core walls, the WCM modeling approach proposed by Beyer et al. (2008) was found to be one the most robust and efficient. The present study modified the proposed WCM according to Arabzadeh and Galal (2017). The number of modules in each segment of the C-shaped wall section was increased to two to provide higher accuracy, as Pelletier and Léger (2017) proposed. This section briefly explains the modeling approach. Arabzadeh and Galal (2017) provided details of the modeling approach.

Geometry of Models

Fig. 3 shows a schematic diagram of the wall model, including the configuration of nodes and elements. Based on the recommendations of Pelletier and Léger (2017), each C-shaped part of the core section was subdivided into three rectangular sections (i.e., the web and two flanges), modeled by displacement-based beam-column elements as piers. The ability to capture the axial load-bending moment interaction in these elements makes them an ideal choice for simulation of RC walls, in which the axial load in each individual element might vary throughout the cyclic excitation. For the non-linear time history analyses, structural elements (i.e., wall segments and columns) were modeled using the fiber section feature in OpenSEES corresponding to the gross section properties of elements, and a fine discretization was selected for each fiber section in the model. Horizontal links running along the weak axis of the sections were used to connect these vertical elements at the story and midstory levels. Full base fixity was assumed for the walls, and the soil-structure interaction was neglected. The slippage in the wall-footing intersections (strain penetration effect), however, was considered by using zero-length elements at the bases of wall segments. The stress-slip model proposed by Zhao and Sritharan (2007) was assigned to the vertical reinforcement, represented by the Bond_SP01 material model in OpenSEES.

As suggested by Stafford-Smith and Girgis (1986), the spacing of the horizontal links was set to provide two rigid links in each story (i.e., four pier elements along the height of the story). In-plane and out-of-plane shear flexibility of the wall segments were considered by defining zero-length elements between the nodes at midheight of

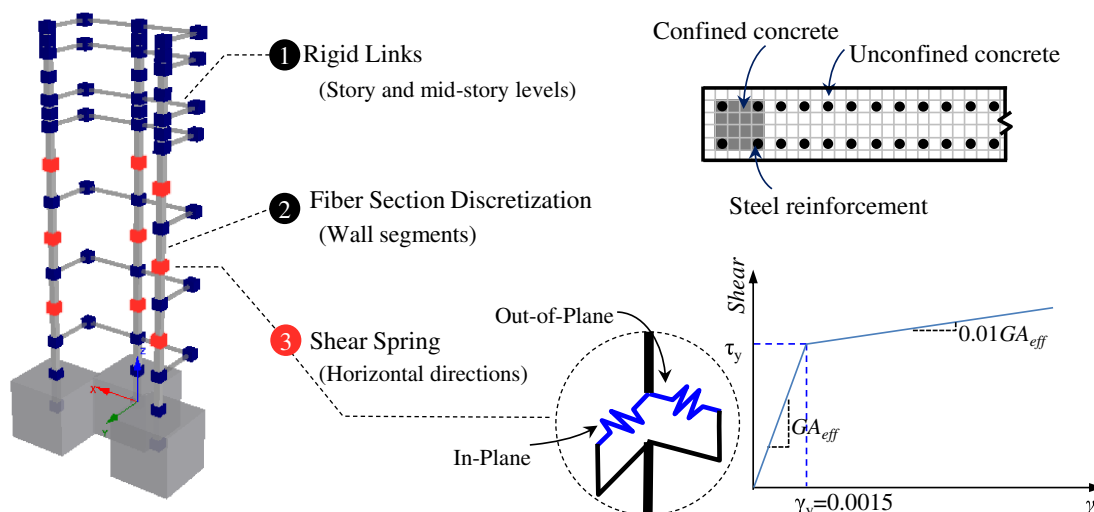


Fig. 3. Model configuration of the RC core wall.

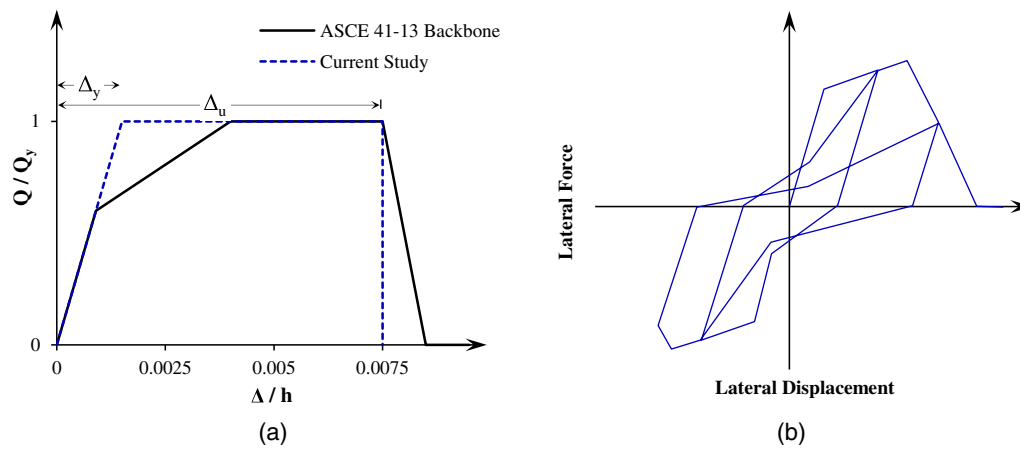


Fig. 4. Reinforced concrete wall response: (a) backbone curve comparison; and (b) typical hysteresis load–displacement curve.

consecutive links. Corresponding stiffness for each translational degree of freedom was assigned, and all other degrees of freedom of the two nodes were slaved using EqualDOF constraints. A bilinear trend was considered for the shear behavior of horizontal zero-length springs in each wall segment [Fig. 3(b)]. Fig. 4 compares the backbone curve for load–displacement response of RC walls proposed by ASCE/SEI 41-17 (ASCE 2017) and the assumptions considered in the current study. A typical hysteresis load–displacement curve for an RC shear wall is also presented in Fig. 4(b). Shear strain at yield was taken as 0.0015 based on test results by Massone et al. (2006) which had good agreement with the values proposed by ASCE 41-17. Moreover, the ultimate drift of 0.0075 considered in the present study is the same as the limit proposed by ASCE 41-17.

Failure Criteria for Materials

Nominal strengths of concrete and steel reinforcement were used in the numerical modeling using the available empirical equations. A modified Kent and Park model (Scott et al. 1982) was used to define the nonlinear constitutive laws for concrete fibers (Concrete02 in OpenSEES). The equations proposed by Mander et al. (1988) were used to consider the confinement effects on the stress–strain behavior of the concrete. The maximum tensile strength of concrete was assumed to be equal to $f'_t = 0.623\sqrt{f'_c}$ (Yassin 1994).

For steel reinforcement, the Giuffré-Menegetto-Pinto hysteretic model (Filippou et al. 1983) with default parameters (Steel02 in

OpenSEES) was adopted to represent the nonlinear behavior of steel bars. No bond-slip was considered with respect to the surrounding concrete.

Table 1 lists the sample material parameters adopted for the verification cases and for the multistory buildings. Fig. 5 shows examples of the cyclic material laws used for the concrete and steel fibers in the numerical model of the specimen MEM-4 from Chen et al. (2016).

Model Validation

In order to validate the capability of the numerical model developed in this study, results of the numerical analysis of two RC specimens subjected to torsional loads were compared with the reported experimental results. This section briefly summarizes each experimental program.

Thin-Walled C-Shaped RC Specimen Tested by Krpan and Collins

Krpan and Collins (1981) tested a thin-walled RC channel beam loaded in pure torsion at midspan. These thin-walled section beams have warping behavior similar to that of RC cores subjected to torsion, and hence were selected as an appropriate case for validating the accuracy of the numerical model. Fig. 6(a) shows the test setup of the experiment. The beam was a 6,400-mm-long C-shaped section torsionally restrained at both ends using rigid concrete blocks and had a 200-mm-thick rigid diaphragm located at midspan.

Table 1. Sample material parameters for the OpenSEES model

Specimen	Parameter	U-shaped beam (Krpan and Collins 1981)		MEM-4 (Chen et al. 2016)		Multistory buildings	
		Unconfined region	Confined region 1	Unconfined region	Confined region 1	Unconfined region	Confined region 1
Concrete material	f'_c (MPa)	−52.00	−59.85	−40.92	−46.40	−30.00	−31.20
	e_0	0.0030	0.0033	0.0027	0.0029	0.0023	0.0024
	K	1	1.151	1	1.134	1	1.040
	f_{cu} (MPa)	−10.40	−11.97	−8.18	−9.28	−6.00	−6.24
	e_u	−0.01	−0.01	−0.01	−0.01	−0.01	−0.01
	f_t (MPa)	4.49	4.82	3.99	4.24	3.41	3.48
	—	#2	#5	ϕ6	ϕ8	10M	25M
Steel material	f_y (MPa)	362	348	277	353	400	400
	f_u (MPa)	490	475	447	573	545	545
	E_s (GPa)	203	194	200	200	200	200

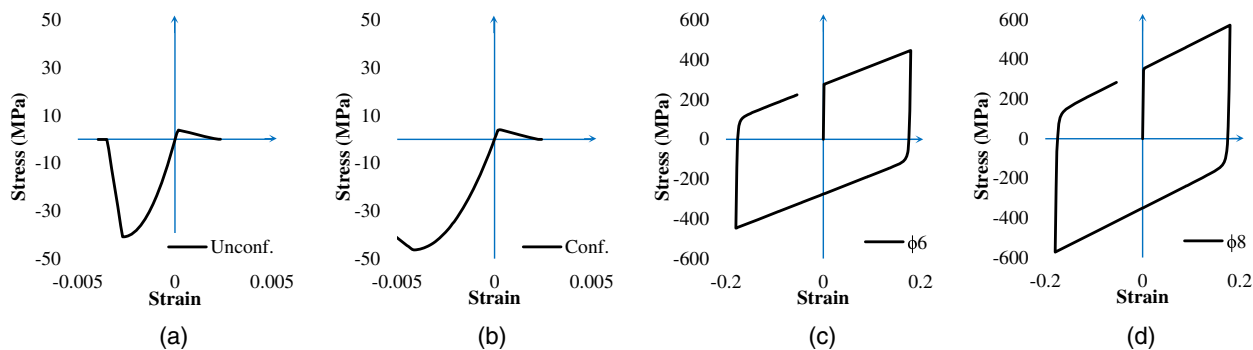


Fig. 5. Sample hysteresis stress-strain behavior of material assigned in the numerical model for specimen MEM-4: (a and b) concrete; and (c and d) steel. (Reprinted from [Chen et al. 2016](#), with permission.)

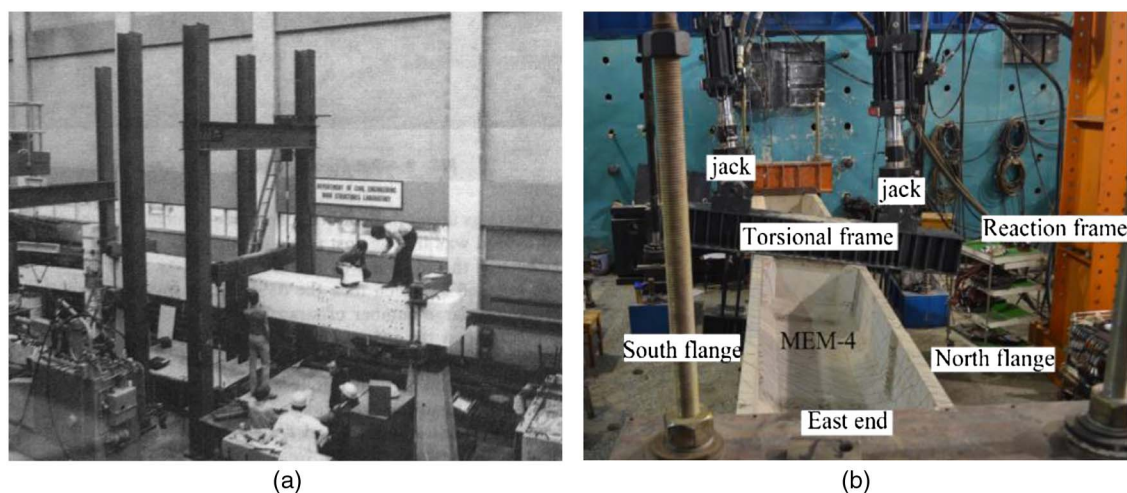


Fig. 6. Experimental test setup. [(a) Reprinted from [Krpan and Collins 1981](#), © ASCE; (b) reprinted from [Chen et al. 2016](#), with permission.]

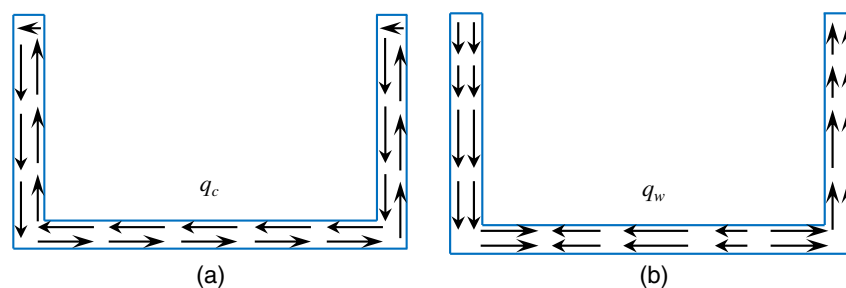


Fig. 7. Shear flow diagrams of a C-shaped section: (a) circulatory torsion; and (b) warping torsion.

Two actuators acted on this diaphragm to create the required torque by applying two equal and opposite forces using a steel loading frame. The beam cross section was 75 mm thick, 840 mm wide, and 650 mm high. The beam reinforcement was designed to avoid brittle shear failures. Number 5 steel bars were used as longitudinal reinforcement and the transverse reinforcement was #2 stirrups at 76 mm.

Fig. 7 shows the shear flow diagrams of a C-shaped section, including circulatory and warping torsion. The first observed cracks during the experiment were vertical flexural type cracks, which occurred at midspan at the bottom of the flange. The cracks were initiated at a torque level of approximately 23 kN · m.

Moreover, yielding of the longitudinal steel bar was first observed at an applied torque of 191 kN · m. Postprocessing of the data recorded from the numerical model (fiber section forces) showed the same trend of crack initiation at the torque level of 36 kN · m. The numerical model determined the steel bars to first yield at an applied torque of 199 kN · m. The final failure of the specimen from the numerical model did not fully match the test results due to the anchorage failure of longitudinal steel during the test. The numerical model assumed the steel bars to be perfectly anchored.

Fig. 8(a) shows the torque-rotation data from the experimental test along with corresponding results from the numerical model. The figure demonstrates good agreement between the experimental

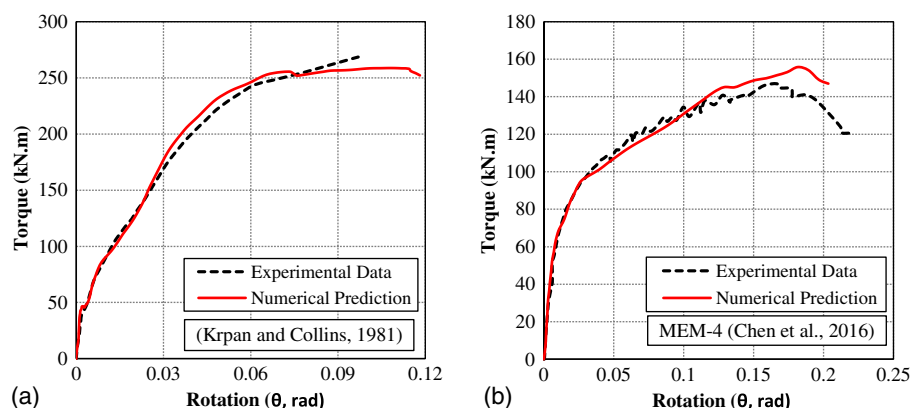


Fig. 8. Comparison of results from the numerical model and the experimental data: (a) specimen tested by Krpan and Collins (1981); and (b) specimen MEM-4 tested by Chen et al. (2016).

records and the numerical predictions prior to longitudinal reinforcement yielding, after which the model was found to have more flexibility than that observed in the experimental test. The small discrepancies between the test results and numerical predictions might be due to the variation of concrete material properties in small-scale specimens and possible measurement errors of the concentrated forces.

Thin-Walled C-Shaped RC Specimen Tested by Chen et al.

Chen et al. (2016) experimentally tested four C-shaped thin-walled RC members of 1:4 reduced scale under pure torsion. Considering the thin-walled feature, the thickness of both the flanges and the web was designed for 70 mm. The specimens were 6,650 mm long, with the height of the flanges and the web width equal to 500 and 900 mm, respectively, aimed to provide a length:thickness ratio of almost 7. The specimens were lengthened by 350 mm on both ends to fix a fastening device which provided a fully anchored situation during the loading process. The specimen MEM-4 selected for the verification purpose in this study was made of 40.92-MPa-strength concrete and was reinforced with 8-mm longitudinal bars ($F_y = 353.3$ MPa) and 6-mm stirrups at 70 mm distances ($F_y = 276.7$ MPa). Fig. 6(b) shows the test setup. Several steel plates were also embedded in both support ends to prevent local failure. A chamfer with a scale of 1:1.125 was considered on the inner side of the connection between flanges and web to reduce the probable stress concentrations. Similar to the work by Krpan and Collins (1981), the torsional load was applied through two opposite and equal forces from two actuators acting on a loading frame, which was fixed on a strengthened diaphragm plate at midspan.

Flexural vertical cracks were first observed at support and midspan when the torque was almost $17 \text{ kN} \cdot \text{m}$, whereas first yielding of the steel bars was measured at a torque level of $85 \text{ kN} \cdot \text{m}$. The numerical predictions followed a similar trend, achieving corresponding torque values of 15 and $93 \text{ kN} \cdot \text{m}$. The numerical model was found to be able to capture the final failure of the specimen as it was observed in the experimental test. The specimen ultimately failed because of the yielding of the longitudinal tensile bar and the crush of compressive concrete as a consequence. There was very good correlation between the torque-rotation curves from the numerical analysis and those from the experimental work of Chen et al. (2016) [Fig. 8(b)]. In larger rotations, the numerical model showed slightly higher twisting resistance than did the experimental model, especially in the postpeak region of the curves.

Table 2. Summary of verification results

Specimen	Results	Longitudinal reinforced yield ($\text{kN} \cdot \text{m}$)	Error (%)	Ultimate capacity ($\text{kN} \cdot \text{m}$)	Error (%)
Krpan and Collins (1981)	Experimental	190	—	268.5	—
	Numerical	186.1	−2	258.7	−4
MEM-4 (Chen et al. 2016)	Experimental	95	—	146.9	—
	Numerical	99.1	4	155.8	6

To evaluate the capability of the numerical model to track the torsional response of C-shaped open section RC members, a quantitative comparison was conducted between the output data resulting from the numerical models and the experimental data. Table 2 summarizes the results from the verification models, including the torque corresponding to the first yield in the longitudinal reinforcement and the maximum torque capacity of the specimen. In general, the inelastic torsional responses were well captured. After the yielding point in both validation cases, yielding of both the longitudinal and transverse reinforcements was observed, from which the load transferred to the reinforcements after the crack initiation can be determined. Hence, the numerical model was found to be capable of tracking the inelastic behavior of thin-walled open section RC walls subjected to torsional forces. The validated model was then used to perform a parametric study of C-shaped RC wall buildings at different levels of torsional irregularity.

Seismic Analysis of Multistory Buildings

Nonlinear time history analyses of the buildings under a group of ground motions were conducted using the macromodel developed in OpenSEES. This section presents configurations of the studied buildings and the selected ground motions and compares the results of NTHA with the data from the response spectrum analysis for each building configuration.

Building Configurations

RC core wall buildings with 8, 12, and 16 stories were analyzed using the nonlinear time history method. The buildings were considered for Site Class C in Montreal, Canada, and were designed according to the National Building Code of Canada (NBCC 2015) and the CSA A23.3-14 (CSA 2014). NBCC (2015) assigns

Site Class C to ground profiles consisting of very dense soils and soft rocks with 360–760 m/s average shear wave velocity, average standard penetration resistance higher than 50, and soil undrained shear strength above 100 kPa.

To evaluate the effect of the torsional loads on the seismic response of the building, each building structure was analyzed at four different levels of torsional sensitivity, $B = 1.3, 1.7, 2.0$, and 2.5 . For the sake of consistency in all the building configurations, the accidental torsion was considered by shifting the center of mass (CM) from the center of rigidity (CR) by a distance equal to $\pm 0.05D_{nx}$. Both Canadian and American design codes classify the torsional sensitivity (irregularity) of building structures. According to NBCC (2015), torsional sensitivity of building structures is defined as the largest ratio of the maximum story displacement at the extreme points of the structure to the average of the displacements at the extreme points in each floor when the structure is subjected to an eccentric equivalent static force with eccentricity of $\pm 0.1D_{nx}$ from the center of mass. A building with $B > 1.7$ is classified as a torsionally sensitive building, and in such a case, the response of the structure cannot be evaluated based on the results of static analysis only, and dynamic analysis is mandatory. However, because the type of dynamic analysis is not specified, RSA is usually the preferred method.

In the multistory buildings studied in this paper, the lateral force resistance was solely provided by an RC core wall. Each floor consisted of five equal spans in both horizontal directions, resulting in approximate outside-to-outside plan dimensions of 25 m (8-story building) and 30 m (12-story and 16-story buildings). Considering the penthouse on the roof, the total heights of the structures were 34.05, 48.65, and 63.25 m for 8-, 12-, and 16-story buildings,

respectively, consisting of a 4.85-m-high first story and a typical story height of 3.65 m. Fig. 9 presents the 3D view and the plan of the 12-story building, as well as the seismic load direction. The core wall system consisted of two C-shaped walls located at the center of the building. The direction of the excitations for the nonlinear response of the building was solely considered to be in the cantilever direction of the walls (Fig. 9). The gravity-resisting system consists of 200-mm-thick flat plate slabs supported by square columns. Table 3 summarizes the main characteristics of the buildings, such as the height, the wall thickness, and outside-to-outside dimensions. The buildings were designed to be built using 30-MPa-strength concrete and 400-MPa-strength steel reinforcement.

For the structural model of the buildings, Boivin and Paultre (2010) observed that a cantilever wall model can reasonably represent the behavior of a complete building model and the stiffness from structural components other than the seismic resisting system has no considerable effect on the wall response predictions. Hence, isolated core wall models can be used for the investigations together with a gravity frame to capture the nonlinear $P-\Delta$ effects. This study, however, developed full 3D models of the buildings, including the gravity columns and story floors, to increase the reliability of the modeling approach in capturing the $P-\Delta$ effects when the structure is subjected to torsional effects (Fig. 9). Hence, the OpenSEES built-in feature for geometric transformation from the basic system to the global coordinate system was used. A similar approach was used for the gravity columns. This approach has been supported by other researchers (e.g., Chae et al. 2012). Structural responses were controlled to avoid any code violation.

The core wall in the original building was designed to have a single plastic hinge at the base of the wall. Thus, the RC wall in the

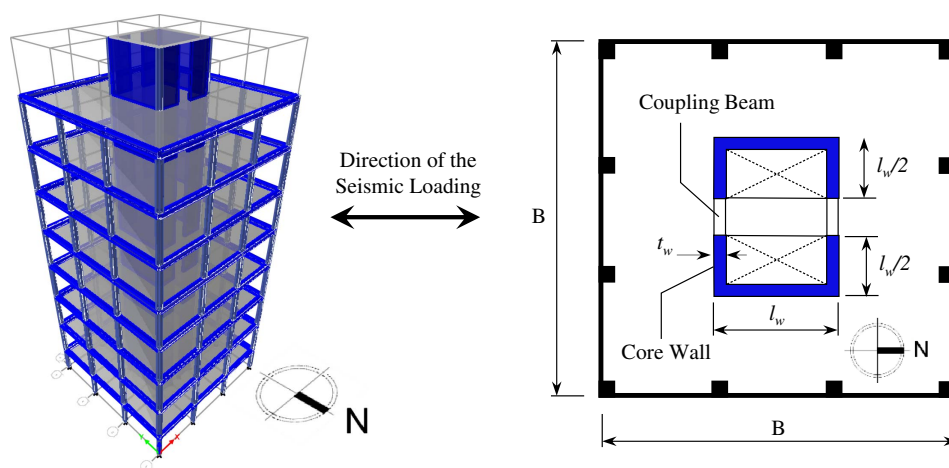


Fig. 9. 3D view and floor plan of studied buildings.

Table 3. Main characteristics of the studied buildings

Characteristic	8-story	12-story	16-story
Typical floor height (m)	3.65	3.65	3.65
Building height (m)	34.05	48.65	63.25
Plan dimension (m)	25	30	30
Core wall length (m)	4×5	5×6	5×6
Core wall thickness (m)	0.3	0.4	0.4
Fundamental natural period of vibration (s) (regular building: $B = 1.3$)	1.34	1.81	2.76
Longitudinal reinforcement ratio at base of wall (%)	1.13	1.19	1.33
Transverse reinforcement ratio at base of wall (%)	0.46	0.56	0.58
Longitudinal reinforcement ratio at midheight of DPH wall (%)	0.54	0.63	0.67
Transverse reinforcement ratio at midheight of DPH wall (%)	0.36	0.34	0.42

first two stories of the 8-story and 12-story buildings and the first three stories of the 16-story building were detailed as a plastic hinge region. The height of the plastic hinge was determined based on clause 21.5.2.1.2 of CSA, which prescribes a plastic hinge length of $0.5l_w + 0.1h_w$, where l_w is the length of the wall in the direction considered and h_w is the height of the wall above the critical section. However, the CSA necessitates the extension of plastic hinge detailing up to $1.5l_w$. Flexural reinforcement was provided according to the CSA bending moment design envelope to satisfy the plastic hinge requirements. This was extended to provide the resisting moment prescribed by CSA.

The shear reinforcement in the assumed plastic hinging region was designed according to CSA, which corresponded to the shear strength required to develop the probable flexural capacity of the wall. The shear detailing assigned for the base plastic hinge was extended in all the buildings to meet the factored shear strength above the plastic hinge height prescribed by CSA, and was lowered to the minimum required reinforcement in the upper floors.

Ground Motion Selection and Scaling

The numerical analyses performed in this study followed the FEMA P695 (FEMA 2009) guideline for seismic assessment of building structures, which prescribes a group of 44 ground motions to achieve reliable results. This study used a group of 48 artificial ground motion records (4×12 records) developed by Atkinson (2009) for time history analysis of the structure due to the lack of records from real earthquakes in eastern Canada. Although the applied method is

acceptable, there are some limitations compared with using historical records or spectrally matched records that need to be acknowledged. For instance, synthetic accelerograms often do not include wave propagation (path effects) or the near-source effects such as rupture propagations. Moreover, for multidirectional excitations, the simulated three components of ground motions are not physically consistent. Thus, time history analyses using artificial records should be done carefully, because of the influence of parameter variability on simulated earthquake results. Hence, it is advisable to use more records and to take into account the results within a limited margin of the mean value. These accelerograms included far-field and near-field records with two different magnitudes, $M = 6$ and $M = 7$. Fig. 10 plots the sample near-field $M = 6$ and $M = 7$ accelerograms used for the NTHA. Table 4 summarizes the selected ground motions characteristics.

Each ground motion was scaled using the ASCE/SEI method (ASCE 2016) such that within the range of $0.2T_1$ and $1.5T_1$, where T_1 is the fundamental period of vibration of the structure, the integration of response spectral accelerations matched or exceeded the integration of target spectral acceleration (i.e., uniform hazard spectrum). The studied buildings were considered for Montreal, Canada only and no seismic hazard variation was considered, which can be noted as a limitation of the current work. The structural damping was considered by means of mass and stiffness-proportional Rayleigh damping. A damping ratio of 5% was assigned, which is a typical value for RC buildings. Although the Rayleigh damping feature in OpenSEES can formulate the damping matrix using the

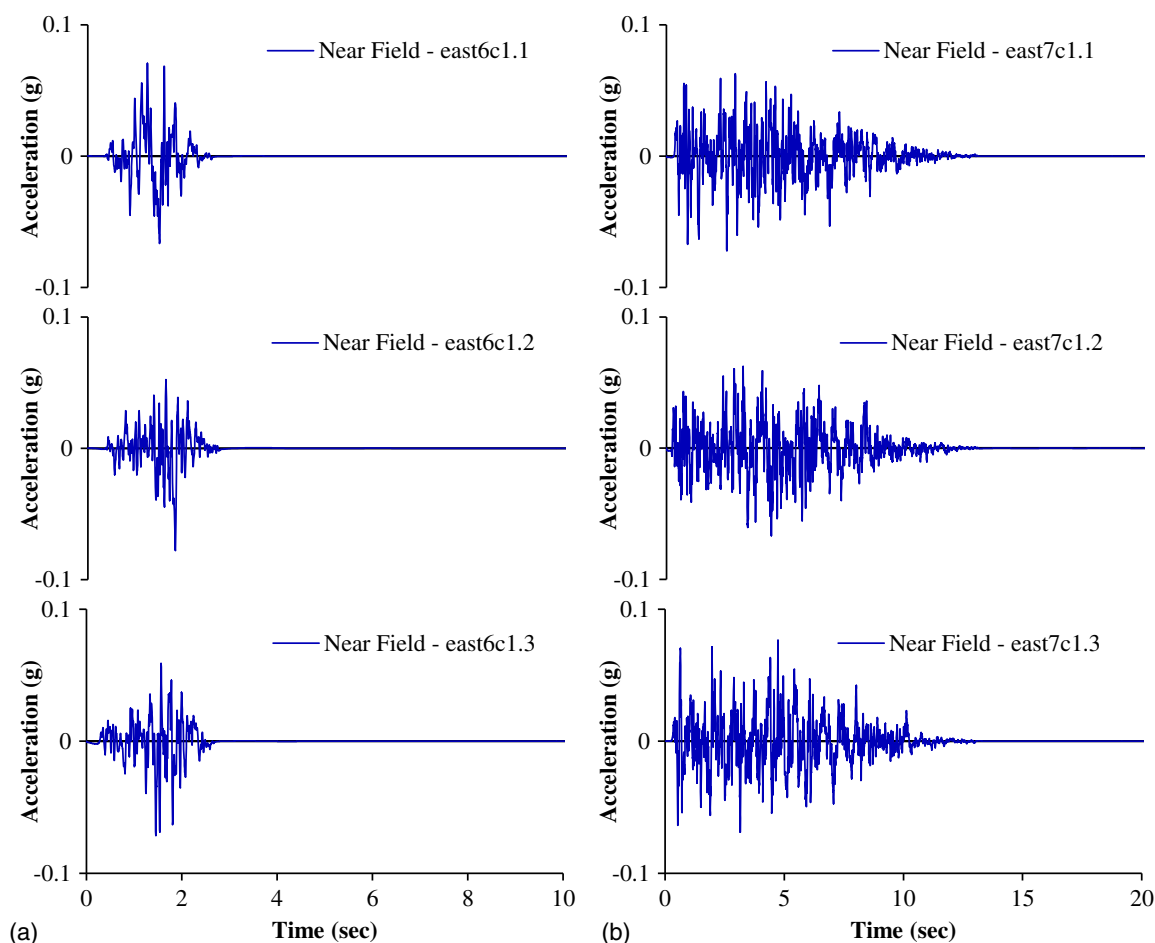


Fig. 10. Sample time histories of selected near-field ground motions: (a) $M = 6$; and (b) $M = 7$.

Table 4. Characteristics of selected ground motions (unscaled)

Near-field ground motions					Far-field ground motions				
Event	M	R_{fault} (km)	PGA (g)	v/a	Event	M	R_{fault} (km)	PGA (g)	v/a
east6c1.1	6	12.8	0.756	0.0467	east6c2.1	6	20.8	0.277	0.0458
east6c1.2	6	12.8	0.768	0.0406	east6c2.2	6	20.8	0.268	0.0266
east6c1.3	6	12.8	0.717	0.0253	east6c2.3	6	20.8	0.295	0.0397
east6c1.4	6	12.5	0.646	0.0455	east6c2.4	6	21.5	0.311	0.0298
east6c1.5	6	12.5	0.597	0.0398	east6c2.5	6	21.5	0.279	0.0295
east6c1.6	6	12.5	0.661	0.0337	east6c2.6	6	21.5	0.192	0.0555
east6c1.7	6	12.8	0.523	0.0573	east6c2.7	6	16.9	0.267	0.0387
east6c1.8	6	12.8	0.565	0.0355	east6c2.8	6	16.9	0.315	0.0404
east6c1.9	6	12.8	0.411	0.0359	east6c2.9	6	16.9	0.287	0.0480
east6c1.10	6	12.8	0.431	0.0363	east6c2.10	6	21.1	0.232	0.0430
east6c1.11	6	12.8	0.405	0.0447	east6c2.11	6	21.1	0.296	0.0361
east6c1.12	6	12.8	0.533	0.0314	east6c2.12	6	21.1	0.218	0.0336
east7c1.1	7	13.8	0.727	0.0525	east7c2.1	7	41.6	0.229	0.0647
east7c1.2	7	13.8	0.598	0.0856	east7c2.2	7	41.6	0.203	0.0567
east7c1.3	7	13.8	0.770	0.0437	east7c2.3	7	41.6	0.229	0.0602
east7c1.4	7	15.3	0.806	0.0570	east7c2.4	7	50.3	0.151	0.0504
east7c1.5	7	15.3	1.162	0.0562	east7c2.5	7	50.3	0.148	0.0759
east7c1.6	7	15.3	0.871	0.0384	east7c2.6	7	50.3	0.122	0.0789
east7c1.7	7	14.2	0.922	0.0738	east7c2.7	7	45.2	0.204	0.0522
east7c1.8	7	14.2	0.764	0.0534	east7c2.8	7	45.2	0.184	0.0539
east7c1.9	7	14.2	1.085	0.0642	east7c2.9	7	45.2	0.180	0.0730
east7c1.10	7	14.9	0.971	0.0551	east7c2.10	7	50.3	0.125	0.0949
east7c1.11	7	14.9	0.971	0.0385	east7c2.11	7	50.3	0.127	0.0610
east7c1.12	7	14.9	0.844	0.0828	east7c2.12	7	50.3	0.122	0.0643

Note: M = magnitude; PGA = peak ground acceleration; R_{fault} = closest distance to fault; v/a = maximum velocity:maximum acceleration ratio.

initial, current, or last committed stiffness matrix, this study used only the initial stiffness matrix. Fig. 11 presents the 5% damped acceleration response spectra of the ground motions.

Results of Time History Analyses

The effect of the torsional sensitivity on the performance of the structures designed based on the Canadian codes (CSA A23.3-14 and NBCC 2015) was investigated using NTHA. After postprocessing the data, time history response plots were created. The results of NTHA and the design envelope obtained from the response spectrum analysis of the structure were compared.

The shear force envelope of the studied buildings was developed to evaluate the effect of torsional sensitivity variation of the

structure on the story shear demand. Fig. 12 shows the results of the NTHA of 12-story buildings as a sample, in which the story shear demands were fluctuated significantly for different ground motion records. As expected, increasing the torsional sensitivity of the structure considerably increased the shear force demand. Moreover, buildings with higher torsional sensitivity exhibited a wider range of variation of the shear force for each story.

Fig. 13 summarizes the results for 8-, 12-, and 16-story buildings, in which the story shear was normalized by the base shear resistance, V_b , specified according to the CSA A23.3-14. By increasing the torsional irregularity of the building, shear demand in some stories of all structures exceeded the shear capacity. This trend continued until at $B = 2.5$, almost all ground motions led to a shear envelope beyond that predicted by the CSA capacity design method. Although

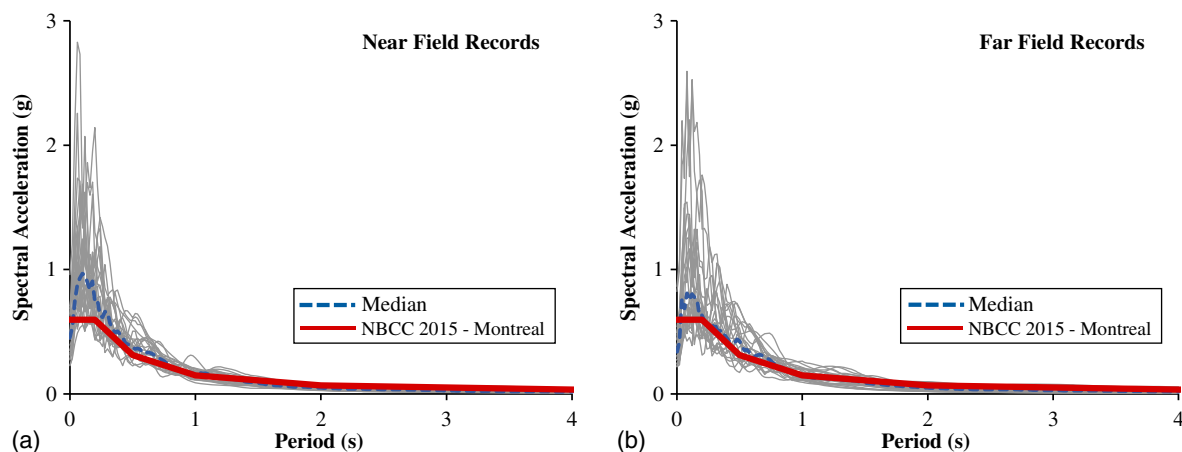


Fig. 11. 5% damped acceleration response spectra of selected ground motions for Montreal.

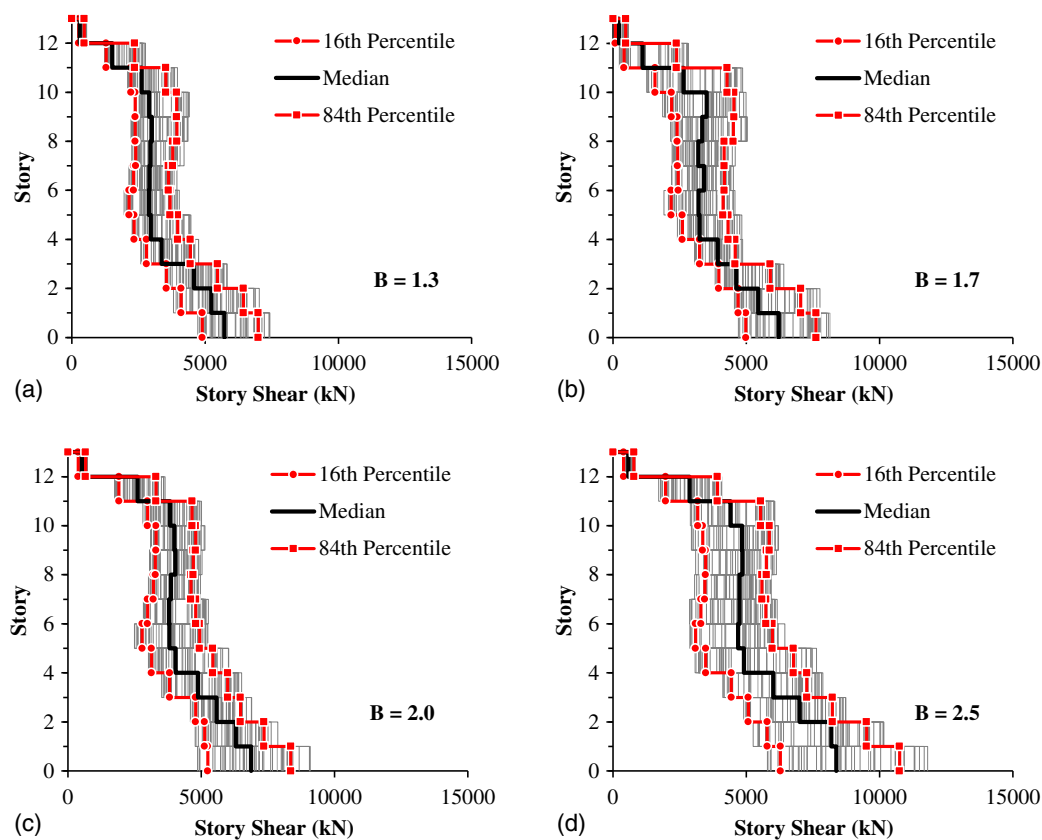


Fig. 12. Sample shear envelopes of 12-story building subjected to individual ground motions.

exceeding the demand because of few artificial ground motion records (i.e., lower torsional sensitivity levels) might not be considered a code contravention, this could be a considerable issue when the capacity design based on the response spectrum analysis fails to resist the shear forces from a major part of the selected earthquakes at higher torsional sensitivity levels.

Comparison of the results from the analysis of buildings with different heights showed that, for a low torsional sensitivity level, the code prediction was more consistent for shorter wall heights (i.e., 8-story building). However, increasing the torsional sensitivity of the building had more-significant effects on shorter buildings than on taller buildings. The peak normalized shear of the 8-story buildings increased 65%, whereas that of the 12-story and 16-story buildings increased 46 and 40%, respectively.

The scope of this study was limited to the buildings in which noticeable contribution of shear deformations in the seismic response of structure is expected. The lateral response of taller buildings is mainly controlled by flexural deformations. Hence, this study excluded buildings over 16 stories. Adebar et al. (2014) compared different proposed methods to consider the contribution of higher modes to the story force demands. They studied 13 building structures of heights up to 50 stories, and the results in terms of the required shear amplification factor showed that almost all the proposed methods result in similar amplification factors for buildings with more than 16 stories.

Moment envelopes along the height of the buildings were also developed to investigate the effect of torsional irregularity of the structure on bending moment demands in the structures. Fig. 14 compares the median moment envelopes for the 12-story building subjected to the selected ground motions. Although shear force demands were highly affected by changing the torsional sensitivity, it was found to have no major effect on the moment envelope

of the building. Pelletier and Léger (2017) reported a similar observation.

Seismic Performance of Core Walls with Dual Plastic Hinge

Panagiotou and Restrepo (2009) introduced a DPH concept for the design of RC shear walls to overcome the disadvantages of extended plasticity and SPH methods, such as special reinforcement detailing. They proved DPH to be an optimized method, because it provides the same control of the performance of the structure as does SPH, while releasing a significant portion of the height of the wall from special reinforcement detailing. Moreover, they found that DPH can result in lower shear force demand in the structure compared with that obtained from buildings with SPH (Panagiotou and Restrepo 2009). Hence, the present study used the DPH method as a solution for controlling the shear force demand in torsionally sensitive structures. All the buildings were designed again according to the DPH concept, and the second plastic hinge was considered at the mid-height of the building, as proposed by Panagiotou and Restrepo (2009). Table 3 presents longitudinal and transverse reinforcement ratios for the base and midheight plastic hinges. The same equation from the CSA was used to determine the height of the plastic hinge at the midheight of the wall, and h_w was considered as the height above the second plastic hinge level, which is equal to half of the total height. However, the plastic hinge detailing was only extended up to the next story level. Moreover, no amplification was applied to the design shear forces for the buildings with DPH; amplification of story shear forces due to the inelastic effects of higher modes was neglected in this case. Fig. 15 presents a sample distribution of reinforcement along the height of the 12-story RC wall with a dual

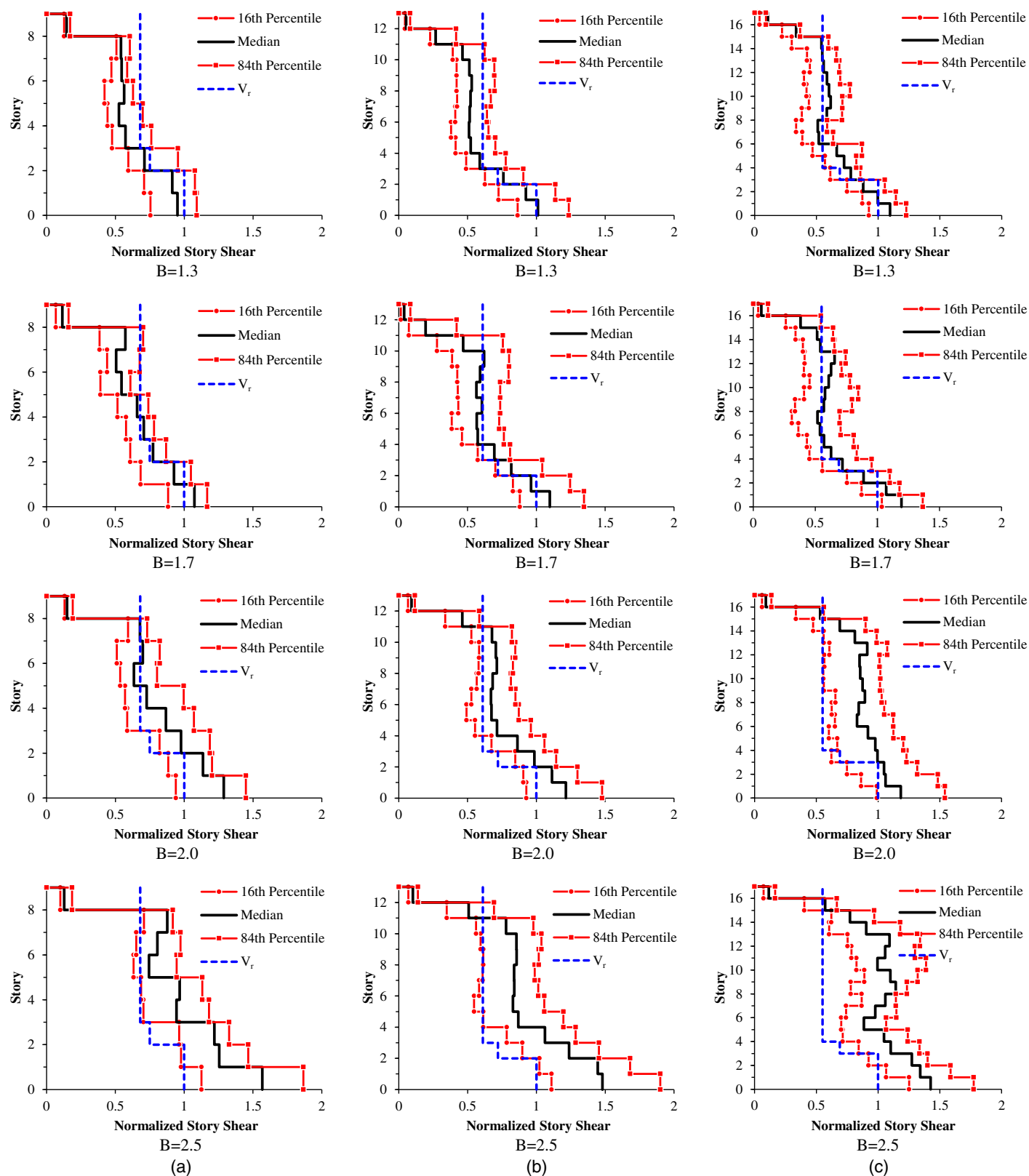


Fig. 13. Evaluation of shear force demand along height of building (NTHA).

plastic hinge. As a comparison, the buildings with SPH had a lower longitudinal reinforcement ratio at the midheight of the wall (i.e., Zone 4 in Fig. 15) than did the DPH walls, which was due to the minimum longitudinal reinforcement required for the plastic hinge zones. Regarding the horizontal reinforcement ratio, the

design of SPH walls based on CSA A23.3-14 considers the shear forces amplified because of the inelastic effects of higher modes. Hence, the ratio of the horizontal reinforcement at the midheight of the wall (i.e., minimum reinforcement according to CSA A23.3-14) to the corresponding value at the base is smaller in SPH walls.

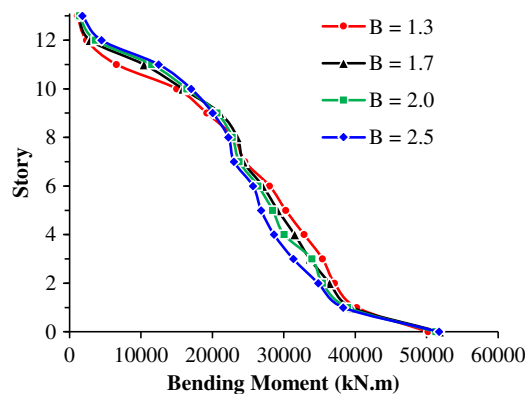


Fig. 14. Median moment envelopes of building structures (NTHA).

Shear Envelopes of Buildings with DPH

Fig. 16 shows the shear force envelopes obtained from NTHA for buildings with DPH. The DPH system performed well in controlling the story shear demand along the height of the structure. Although in some stories the shear demand still exceeded the shear resistance provided based on the capacity design method, the enhancement of the performance even in torsionally irregular buildings is noticeable. Considering the median of the results from the 48 selected ground motions, all the buildings were found to provide capacities higher than the shear force demand except in Stories 3–4 and 3–5 for the 12-story and 16-story buildings, respectively. However, the demand:capacity ratios were limited to 1.12 and 1.11, respectively. By increasing the torsional sensitivity of the building, the excessive demand predicted for the 8-story structure

ranged from 5 to 11%. The predicted range was from 6 to 17% and 6 to 15% on average for the 12-story and 16-story buildings, respectively. Although excessive shear demand was predicted to occur even in the top stories of buildings with the SPH system, especially when the torsional sensitivity of the structure increased beyond $B = 2.0$, none of the buildings with the DPH system experienced a demand:capacity ratio greater than 1.0 in the upper half of the building. The results for DPH buildings, however, showed shear demands beyond the shear resistance in torsionally sensitive buildings ($B \geq 1.7$), but quite limited (within 6–15% average in 4–6 midheight stories) compared with the average of 9 to 53% observed in the buildings with the SPH (i.e., average 3–38% smaller shear demand:capacity ratio).

Response Comparison for SPH and DPH Core Wall Buildings

As observed in the previous section, the overall performance of DPH design in controlling the shear force demand along the height of the structures was significant. The two design concepts, SPH and DPH, were compared in terms of shear demand:capacity ratio of each floor.

Fig. 17 presents the results of the comparison for the 12-story buildings. The NTHA of structures designed based on the DPH concept resulted in shear demand:capacity ratios of less than 1.0 in most stories, and only a few stories in the irregular structures (mainly at $B \geq 2.0$) suffered from underestimation of the shear demand by the RSA. Moreover, developed box plots showed that the fluctuation of results of NTHA of buildings with DPH subjected to the 48 individual records was considerably less than that of buildings with SPH.

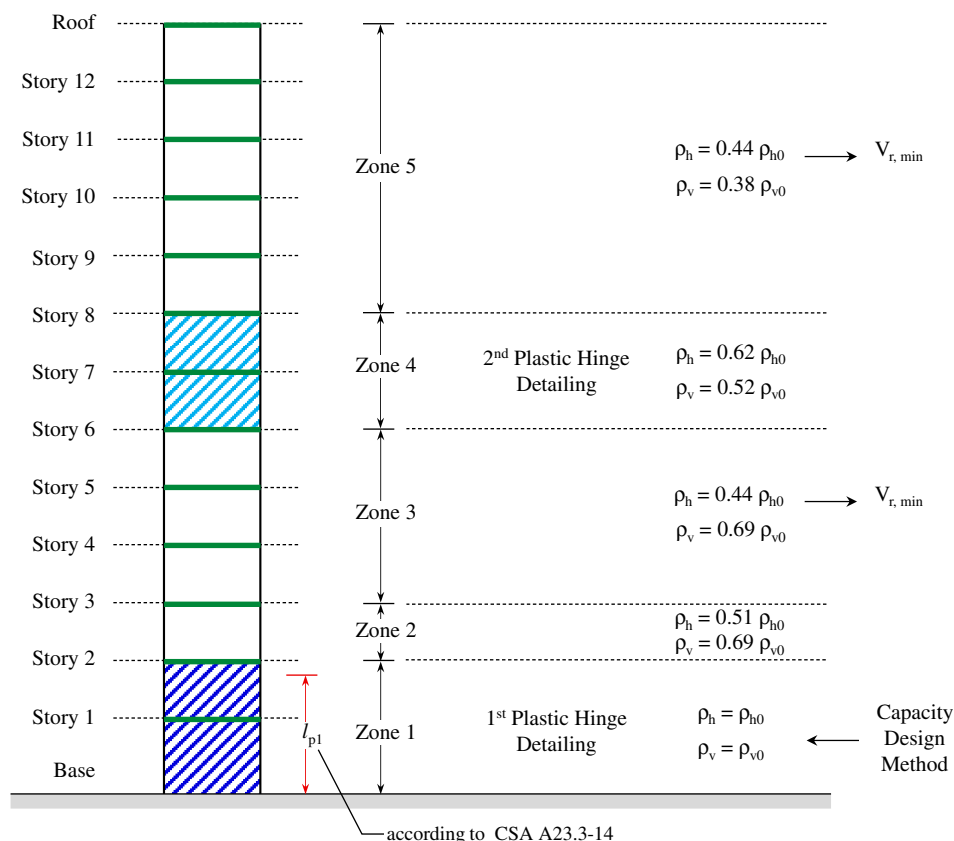


Fig. 15. Sample reinforcement ratios along height of 12-story RC wall with dual plastic hinge.

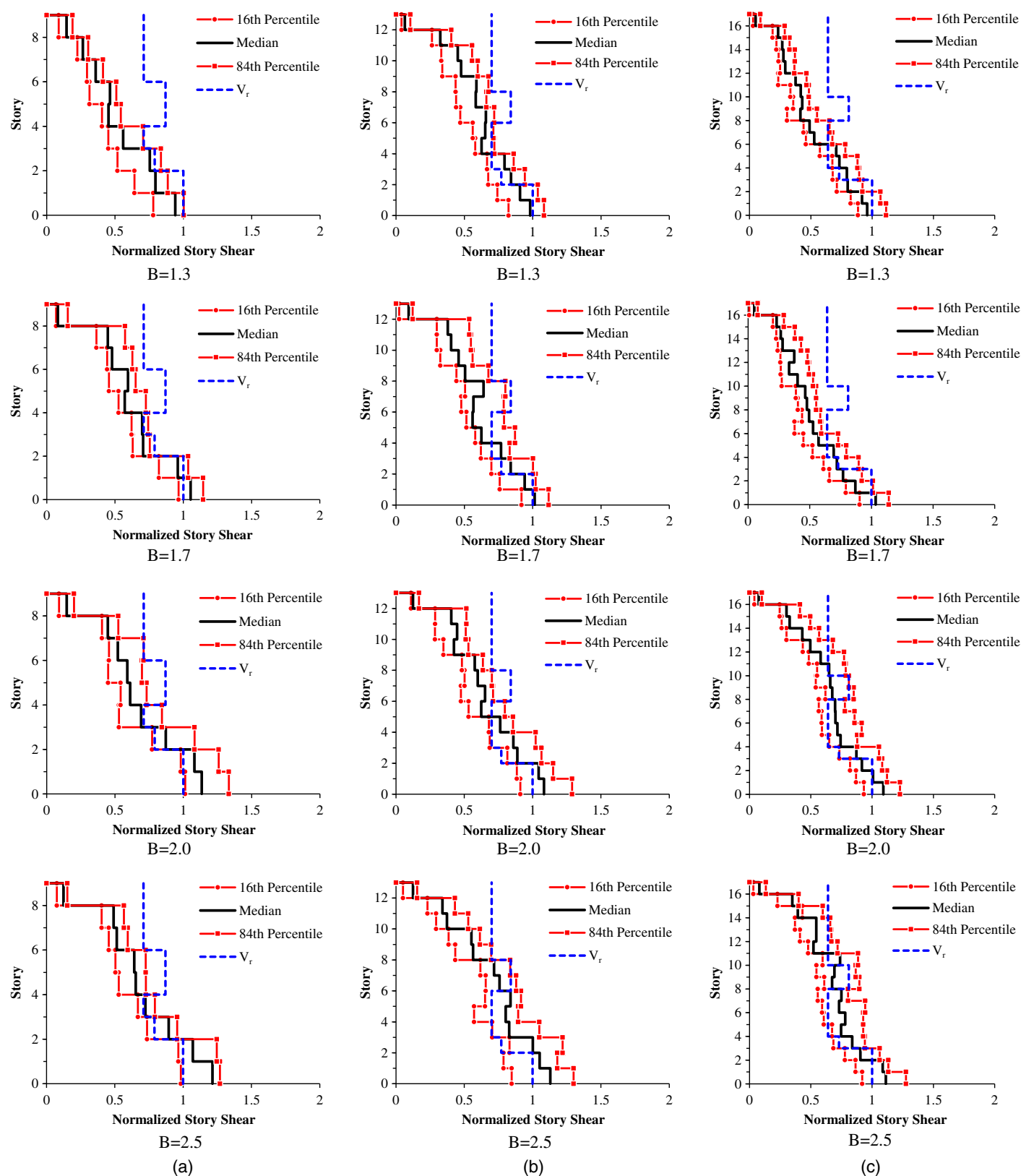


Fig. 16. Shear force demand along height of building with DPH (NTHA).

Similar results were observed for 8-story and 16-story buildings. Table 5 summarizes the seismic assessment results for RC cores and compares the normalized shear demand:capacity at the base of the walls in all the buildings. The same trends of results were captured in 8-story and 16-story structures. This promotes the DPH concept as

an efficient alternative for the design of torsionally sensitive buildings. A quantitative comparison also showed up to 65, 47, and 30% increases in the normalized base shear demand (normalized to V_r) by increasing the torsional sensitivity in 8-, 12-, and 16-story SPH buildings, respectively. The corresponding increases in DPH

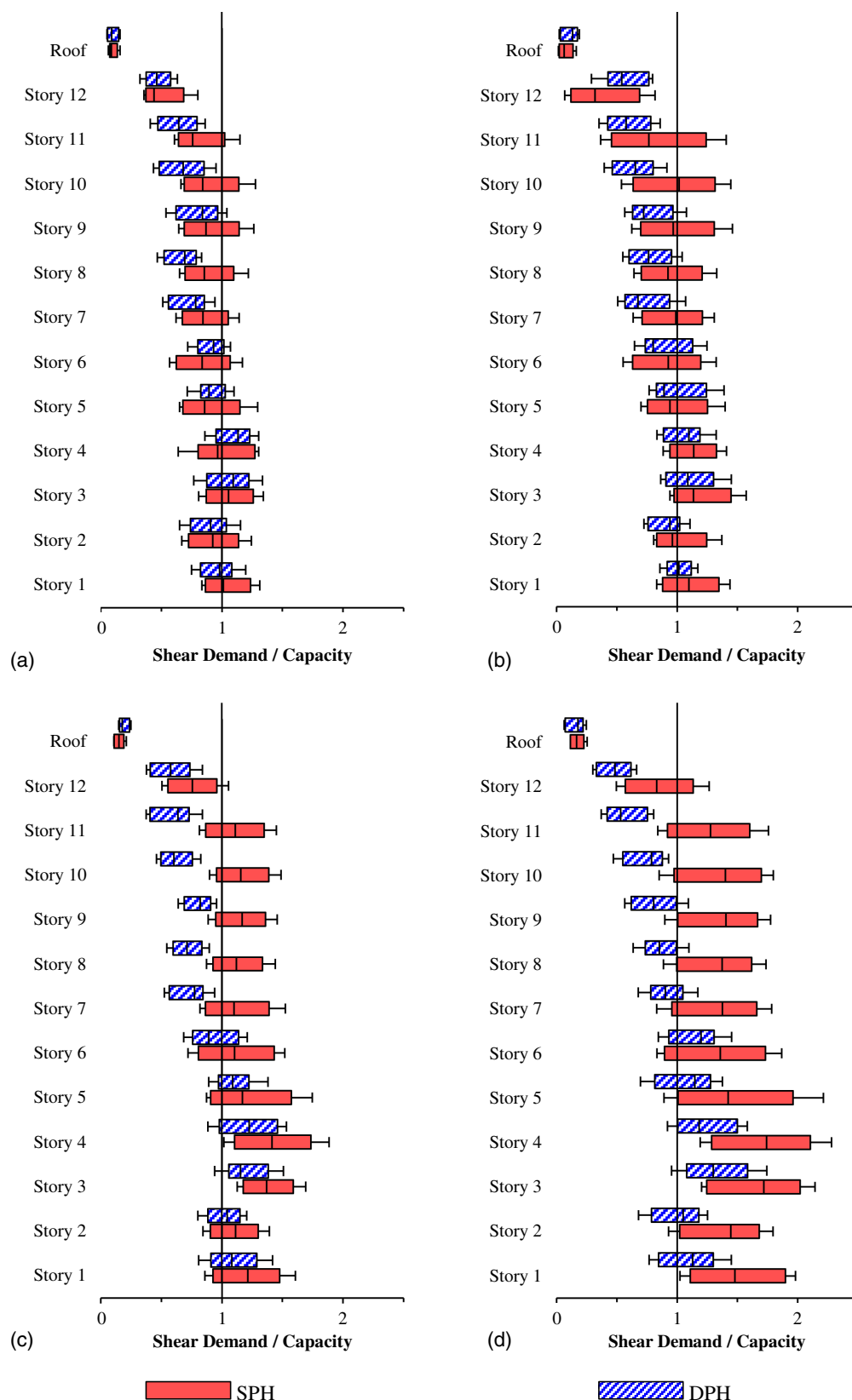


Fig. 17. Shear demand:capacity ratios for 12-story buildings with SPH and DPH.

buildings were up 29, 15, and 16%, respectively. Generally, taller SPH buildings showed larger normalized shear demand, but had fewer discrepancies because of torsional sensitivity variations. This was quite expected because the shorter buildings were subjected higher levels of T/M ratios along the height of the structure, and

torsional effects were greater in these buildings. Buildings with DPH showed opposite responses with increasing height of the structures, because of the influence of the second plastic hinge on the higher modes of vibration of the structures. In fact, DPH had greater mitigating effects on the higher modes of taller buildings, and hence

Table 5. Summary of seismic assessment results for RC cores

Torsional sensitivity, B	V_{base}^a/V_r (SPH)			V_{base}^a/V_r (DPH)		
	8-story	12-story	16-story	8-story	12-story	16-story
1.3	0.95	1.01	1.09	0.94	0.98	0.96
1.7	1.07	1.10	1.19	1.05	1.01	1.03
2.0	1.29	1.21	1.18	1.13	1.08	1.09
2.5	1.57	1.48	1.42	1.21	1.13	1.11

^aMedian value of 48 ground motions.

Table 6. Required enhancement in dynamic shear amplification factor

Torsional sensitivity, B	Flexural yielding	8-story		12-story		16-story	
		Base	Average	Base	Average	Base	Average
1.3	SPH	—	—	1.04	1.04	1.12	1.18
	DPH	—	—	—	—	—	1.15
1.7	SPH	1.12	1.11	1.11	1.07	1.26	1.42
	DPH	1.09	1.09	1.03	1.03	1.08	1.11
2.0	SPH	1.32	1.19	1.26	1.19	1.35	1.55
	DPH	1.18	1.16	1.13	1.12	1.12	1.10
2.5	SPH	1.64	1.36	1.56	1.42	1.45	1.75
	DPH	1.28	1.17	1.19	1.15	1.12	1.14

resulted in a limited contribution of higher modes to increasing the story shear force demand in the structures.

Discussion

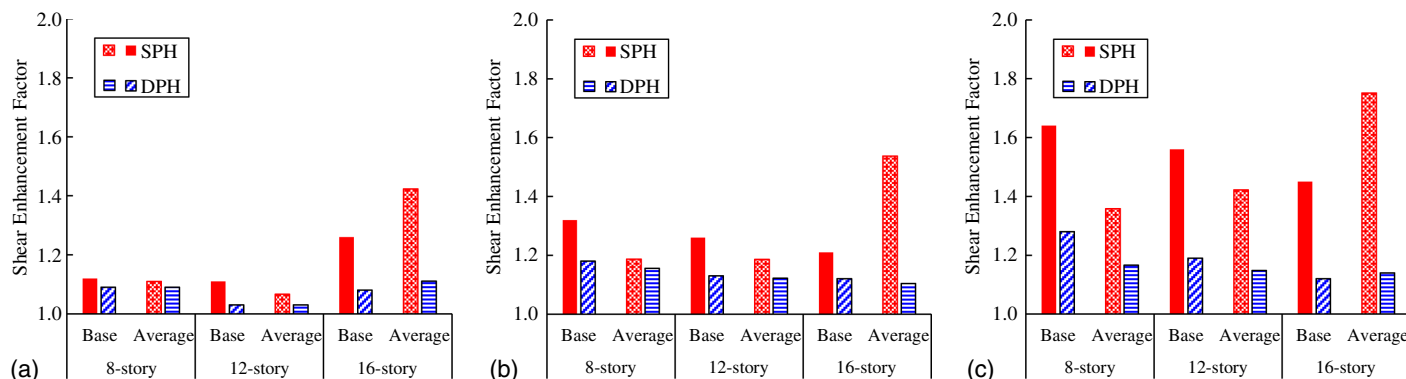
Subject to the limitations imposed by the underlying assumptions, the results of the study in the previous sections show that the shear force demand predicted by the capacity design method is inadequate in some cases even when using the dynamic shear amplification (DSA) prescribed by CSA A23.3-14. As a result, the shear demand envelope obtained needs to be enhanced further to address the issue. In the explanatory notes, the Cement Association of Canada (CAC) *Concrete Design Handbook* (CAC 2016) explicitly states that the shear amplification factors adopted in the 2014 version of CSA A23.3 is a lower-bound value compared with the recommendations by Ambrose et al. (2013). The shear amplification prescribed was adjusted for spectral shapes in eastern Canada based on the work of Paultre (Ambrose et al. 2013). Table 6 lists the required enhancement in terms of a percentage of the code amplification factor for the different building configurations studied here. Excepting the torsionally regular buildings ($B < 1.7$), in most of which the DSA prescribed by CSA 23.3-14 performed well by reasonable estimation of shear force demand, the NTHA of

torsionally sensitive buildings resulted in shear envelopes that exceeded the amplified envelope predicted by RSA by 3–64% and 3–75% for SPH and DPH buildings, respectively (Fig. 18). No constant trend existed between the results for lightly irregular buildings ($B \approx 1.7$). For the buildings with $B \geq 2.0$, however, the base shear enhancement factor decreased by about 10% by increasing the height of the SPH structures. Moreover, an ascending trend was observed for the average story shear enhancement factor when the number of stories increases, resulting in up to a 29% increase. Results for the DPH buildings, however, showed descending trends for both the base and average story shear enhancement factor, 9 and 4% decreases, respectively. Hence, the DPH concept was found to be an efficient alternative when high torsional sensitivity in the structural response is expected.

Conclusions

This study provides results from nonlinear time history analysis of C-shaped RC walls to investigate the performance of response spectrum analysis for the design of RC building structures with an RC core wall. A macromodeling approach using the wide column analogy was used to develop and calibrate a numerical model. A total of 1,152 nonlinear time history analyses were conducted on 8-, 12-, and 16-story buildings using a group of 48 artificial ground motion records. The buildings were designed for Montreal, Canada using the response spectrum analysis method and according to the Canadian design codes, NBCC 2015 and CSA A23.3-14. Results showed that

- Although RSA is a robust and practical dynamic analysis method, results of NTHAs of torsionally sensitive buildings showed notably larger story shear demand than that predicted by RSA using the NBCC (2015) provisions. The predictions were more consistent for regular low-rise walls ($B < 1.7$) because of lower T/M ratios; however, a substantial increase of up to 65% was observed in the peak base shear when the torsional sensitivity of the structure reached $B = 2.5$. Results are in agreement with previous findings in the literature (Tso and Yao 1994; Pelletier and Léger 2017).
- A significant contribution of higher modes noticeably affects the story force demand in the buildings as torsional irregularity of the building increases, especially in shorter buildings. This is expected because in torsionally irregular buildings, even third and fourth modes of vibration substantially usually affect the seismic response, whereas the seismic response of regular buildings is usually controlled by only the first and second modes. It was also shown that the range of variations increases with the increase of the torsional sensitivity of the structure.

**Fig. 18.** Required enhancements in shear amplification factor.

- More-effective mitigation of higher mode effects was observed in DPH buildings, which resulted in lower shear force demands compared with SPH buildings. The DPH method was found to be an efficient alternative for controlling the shear force demand in torsionally sensitive buildings. Using the DPH method, the extra shear demands observed in the torsionally irregular SPH buildings was controlled by 14% and 36%, corresponding to 16 and 8-story buildings, respectively. However, the shear demand envelopes were still beyond the RSA predictions in high torsionally irregular buildings ($B > 2.0$). Moreover, the fluctuation of the shear force demands was lower than the corresponding values in the SPH buildings.
- The new dynamic shear amplification factor in CSA A23.3-14 provisions performed well in predicting the shear force demand required for design of torsionally regular RC walls ($B < 1.7$) using the capacity design method and RSA analysis. However, results of RSA analysis based on the response spectrum prescribed by NBCC (2015) need to be further amplified as the torsional irregularity increases. This study proposed shear force enhancement factors, and showed that 3–64% and 3–75% increases are required for SPH and DPH buildings, respectively.
- By increasing the torsional sensitivity of SPH structures, the required shear enhancement factor was decreased for base shear force, whereas it was increased for the average story shear force. In DPH structures, however, a descending trend was observed for both the base and average story shear forces.

Acknowledgments

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