

Seismic Collapse Risk Assessment and FRP Retrofitting of RC Coupled C-Shaped Core Walls Using the FEMA P695 Methodology

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Abstract: Despite the frequent use of C-shaped reinforced concrete (RC) cores as the primary force-resisting system in multistory buildings, there are still challenges in estimating their inelastic seismic response, especially when they are used as a coupled wall system. Recent studies showed the inadequacy of old code provisions in predicting the seismic shear demands of these systems. This means that many existing RC cores are structurally deficient and need to be retrofitted. One alternative is to retrofit RC shear walls using FRP composite materials to enhance the capacity and ductility of the system. The current paper focuses on two aspects of coupled C-shaped RC core systems: (1) seismic collapse of the system for different torsional sensitivities, and (2) effect of FRP retrofitting on the seismic response of the structure. Modifications are proposed to the wide column model recently proposed by other researchers to accurately capture the inelastic torsional behavior of RC cores, including different modes of failure. Moreover, a simplified spring model is proposed to consider the effect of fiber reinforced polymer (FRP) retrofitting with vertical strips along with X-bracing. The proposed modeling approach is validated against available experimental data. Nonlinear incremental dynamic analysis (IDA) of a typical 12-story RC building structure located in Eastern North America was performed using *OpenSEES*, following the FEMA P695 methodology. It was shown that, although torsional sensitivity has no significant effect on the interstory drift ratios of the building, it can significantly decrease the collapse margin ratio (CMR). Combined shear/flexural failure was found to be the most common failure mode. Observed results also confirmed that FRP strengthening can be used as an efficient method for enhancing the collapse resistance of RC core wall systems. By using a proper strengthening scheme with FRP material, a more than 60% increase in the CMR can be achieved for the structural system. DOI: 10.1061/(ASCE)ST.1943-541X.0001820. © 2017 American Society of Civil Engineers.

Author keywords: Risk assessment; Reinforced concrete (RC); C-shaped core; Seismic; Concrete and masonry structures.

Introduction

Reinforced concrete (RC) walls are often used as coupled systems in the construction of multistory buildings because of their advantages in comparison with individual walls such as higher lateral stiffness, lower bending moments on each individual wall, and higher energy dissipation because of the inelastic deformations of coupling beams (El-Tawil et al. 2010). These elements have been extensively used in medium-rise and high-rise building structures in the past decades. Nowadays, RC coupled shear walls are popular lateral force resisting systems, especially in high-risk seismic zones (Farhidzadeh et al. 2013). The reason behind this trend is that RC coupled walls are significantly capable of controlling the interstory drift ratio, which has been frequently used as a performance indicator in the design of structures (Carrillo and Alcocer 2012). Similarly, these structural systems are quite efficient in reducing the associated implication of nonstructural element damage.

The expected energy dissipation mechanism of a ductile RC wall system under lateral deformations is flexural yielding (i.e., plastic hinges) at the base of both the cantilever and the coupled wall systems and at both ends of each coupling beam in a coupled wall system (Boivin and Paultre 2012). A series of design provisions are specified in the current codes to confine the inelastic response at the wall base. These are aimed at ensuring enough strength against undesirable modes of failure like brittle shear failure (Ghorbanireneni et al. 2012).

Many researchers have conducted both experimental and analytical investigations to identify the behavior of coupled walls and to improve the performance of these systems. The C-shaped coupled wall system (i.e., core wall) is one of the simplest arrangement and is popular in practice. Despite its popularity, however, there have been relatively few studies on the seismic behavior of this RC structure (Beyer et al. 2008a), necessitating research on the seismic performance of C-shaped cores. One of the most important characteristics of these nonplanar wall systems is their response when the structure is subjected to torsional effects due to the eccentricity of lateral forces. This is more significant when the structural system is asymmetric in plan regarding lateral stiffness and strength distribution. Such a configuration in the plan of a building is prone to large torsional response during a severe earthquake. Reports and field observations after past earthquakes showed severe structural damages because of torsional effects (Hart 1975; Esteva 1987). A recent investigation by Dizhur et al. (2011) reported significant structural damages, apparently caused by a “torsionally sensitive response,” after the Christchurch earthquake in 2011.

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Note. This manuscript was submitted on June 30, 2016; approved on February 21, 2017; published online on May 26, 2017. Discussion period open until October 26, 2017; separate discussions must be submitted for individual papers. This paper is part of the *Journal of Structural Engineering*, © ASCE, ISSN 0733-9445.

Most of the research carried out in the past focused on the behavior of planar RC walls, including various proposed approaches for predicting their nonlinear flexure-shear interaction behavior (Colotti 1993; Elwood 2002; Massone et al. 2006, 2009; Mullapudi and Ayoub 2009; Zhang and Xu 2009; Jiang and Kurama 2010; Beyer et al. 2011; Panagiotou and Restrepo 2011; Fischinger et al. 2012). These approaches were mostly based on fiber-section elements, such as the multiple vertical line elements (MVLEs) proposed by Vulcano et al. (1988). The biaxial behavior of concrete material [e.g., modified compression field theory (Vecchio and Collins 1986)] was also considered in some of these approaches.

In contrast, experimental research on the performance of nonplanar (e.g., C-shaped) RC walls subjected to lateral loads is very limited. In one study, Ile and Reynouard (2005) examined three full-scale U-shaped RC walls under cyclic lateral loading. The purpose of the study was to investigate the behavior of U-shaped walls against uniaxial and biaxial bending and shear and to compare the design provisions required by two versions of Eurocode 8 (CEN 2004). A shell model was also developed for simulation of nonplanar RC walls. Beyer et al. (2008a) investigated the bidirectional quasi-static cyclic response of ductile U-shaped RC walls by conducting experimental tests on two half-scale specimens with different thicknesses. The tests mainly focused on the flexural behavior of walls, considering different directions of loading (two orthogonal and one diagonal). Results showed the diagonal direction as the most critical direction, in which the maximum moment resisted by the wall was less than the corresponding value calculated by the plastic hinge analysis. Moreover, the displacement capacity of the wall in the diagonal direction was found to be smaller than that in the orthogonal directions. A simplified numerical model was also developed by Beyer et al. (2008b), who proposed a practical approach for implementation and analysis of U-shaped walls. The numerical approach was based on a wide column analogy, and has been shown to produce reasonable estimation of the inelastic displacement response for slender walls. Constantin and Beyer (2012) used a three dimensional (3D) multilayered shell element model for U-shaped walls to capture their local and global behavior under diagonal loading. The model was developed using the software *VecTor4* developed at the University of Toronto and was found to be accurate for loading capacity of the wall but not for its displacement ductility.

Lowes et al. (2013) examined three 1/3-scale C-shaped wall specimens, representing part of a coupled RC core system, under biaxial loading protocols. Results of cyclic tests showed that bidirectional loading significantly affected the response for displacement cycles in excess of the yield displacement. At these displacement levels, bidirectional loading resulted in a significant reduction in the stiffness of the wall in the direction parallel to the web of the wall (loading activating strong axis bending).

Recently, Lu and Panagiotou (2014) presented a 3D cyclic model for nonplanar RC walls based on a beam truss analogy. The model was able to predict the effects of flexure-shear interaction, considering biaxial behavior of concrete material, and account for mesh size effects. Although the proposed model has been revised several times and has been validated for three RC T-shaped, C-shaped, and I-shaped section wall specimens, the modeling approach was found to be complicated in terms of calibration of truss members and material properties (Kolozyari 2013). The results were also sensitive to precise tracking of the displacement responses of walls in a wide amplitude.

In design of RC shear walls, the fundamental design equations are mainly based on the “plane sections remain plane” assumption, which is unable to capture the shear lag effects related to flexure

and warping torsion. Such effects can be substantial in nonplanar (C-, I- or T-shaped) wall configurations, and might affect the response of a structural system in seismic excitations. Boivin and Paultre (2010) showed that the seismic provisions proposed by the National Building Code of Canada (NBCC 2005) and CSA A23.3-04 (CSA 2004) for design of ductile RC shear wall buildings could considerably underestimate shear demand, especially at the base of the shear wall system. This issue was due to the fact that the amplification effects from the higher modes of vibration could not be efficiently taken into account by current capacity design methods. In seismic design of a multistory ductile RC wall, this could result in a considerable underestimation of the seismic force demand. For this reason, more studies need to be conducted on the seismic performance of these structural systems and the effectiveness of available retrofitting methods, both of which were investigated in the current study. Recent research on the seismic design of RC shear walls (Pelletier and Léger 2017) showed that a more reliable shear force demand can be achieved by using the recently introduced dynamic shear amplification factor from CSA A23.3-14 (CSA 2014). This factor should be applied to account for the inelastic effects of higher modes and to prevent brittle failure. However, RC shear wall systems that are designed based on CSA A23-04 need to be controlled for shear demands, and CSA A23.3-14 excludes coupled and partially coupled walls from the clause “accounting for inelastic effects of higher modes.” Furthermore, NBCC (2015) provides a higher mode factor M_b , which is equal to 1.0 for coupled shear walls except in very occasional cases, $T_a = 2.0$ s and $S_{0.2}/S_5 = 65$, in which M_b is equal to 1.03. In contrast, it was found by Boivin and Paultre (2010) that the shear envelope calculated based on the capacity design method is significantly unconservative in either the cantilever or coupled wall directions. Therefore, more accurate evaluations for future designs and retrofit options for existing building are essential.

FEMA P695 (FEMA 2009) has proposed a methodology to assess the seismic performance of structures. This methodology provides a rational method for evaluating the adequacy of proposed response parameters, including the response modification factor (R), the system overstrength factor (Ω_0), and deflection amplification factor (C_d) of a seismic force-resisting system. Using nonlinear analysis techniques, the methodology combines certain steps to develop and analyze a specific structural model for probabilistic collapse assessment of a structure. The methodology explicitly considers uncertainties in selected ground motions, modeling approach, design, and test data.

Because of the lack of an extensive database of detailed experimental data for nonplanar RC walls in the literature, among the modeling approaches available for predicting the response of these systems, only a few have the capability of tracking the cyclic response of C-shaped RC cores when subjected to modest to significant shear-flexure interaction (SFI). In some models, more studies have to be conducted on the sensitivity of outputs to input parameters (e.g., material properties and modeling assumptions), and more validation against global behavior (load displacement) and local responses (rotations, stresses, strains) is needed. Hence, the WCM proposed by Beyer et al. (2008b), which incorporates coupling effects between axial/flexural and shear responses under cyclic loading conditions, was found to be the most practical modeling approach in this case because of its simplicity of use and the accurate results captured by the model.

The objective of this paper is to quantitatively evaluate the seismic collapse risk of coupled RC core walls with different torsional sensitivity factors, and to assess the effect of FRP retrofitting of these structural systems on their seismic responses. This evaluation is done according to the FEMA P695 methodology.

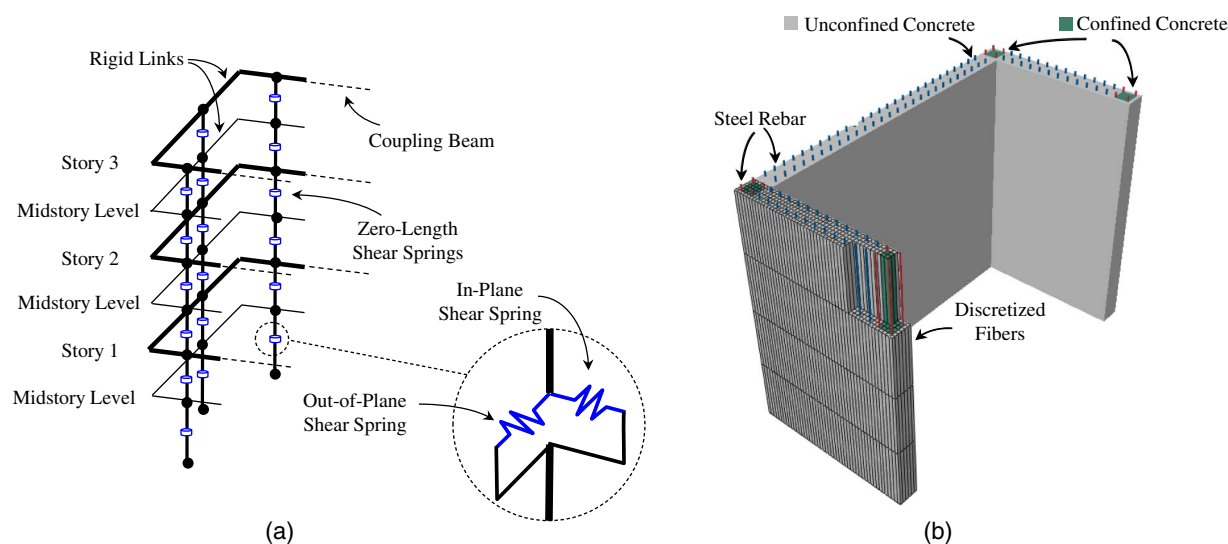


Fig. 1. Wide column model for the C-shaped core wall: (a) isometric view of RC core elements and position of the wall elements and links; (b) subdivision of the fiber cross section of the C-shaped RC core

A modified wide column model (WCM) was proposed for simulating the nonlinear response of RC cores considering torsional effects. A simplified 3D macromodel was developed using *Open System of Earthquake Engineering Simulation (OpenSEES)* (McKenna et al. 2013); subsequently experimental test data from the literature were used to validate the accuracy of the modeling approach. Moreover, a simplified spring model was proposed to consider the effectiveness of using FRP wrapping, vertical strips, and X-bracing for retrofitting purposes. After validating the numerical model, nonlinear incremental dynamic analysis (IDA) of a typical 12-story RC structure located in eastern Canada was performed, following the FEMA P695 methodology. A series of 50 artificial ground motions proposed by Atkinson (2009) were used for IDA analyses of the RC core in different levels of torsional sensitivity. The analyses results were compared to determine the dynamic collapse capacity of the structure before and after FRP retrofitting.

Analysis Methodology

Several analytical models have been developed in the literature to capture the response of shear/core walls. However, when the nonlinear response is of interest, the behavior of these C-shaped RC cores is significantly affected by the interaction between axial force, flexure, and shear. Precise capturing of these effects presents essential difficulties in macroscale numerical modeling (Ile and Reynouard 2005). Although the adoption of the model according to cyclic material properties (microscale modeling) can lead to more accurate results in this case, more numerical effort is a major drawback. Hence, in the current study a macromodeling method was selected for numerical analysis of C-shaped core systems, using the most accurate boundary conditions and constraints. The numerical modeling in this paper is based on a WCM proposed by Beyer et al. (2008b) for inelastic analysis of C-shaped walls. Though Pelletier and Léger (2017) showed that the accuracy of numerical results could be improved by increasing the number of modules in the WCM, especially for high ductility levels, numerical analysis requires more computational effort and presents implementation difficulties. Hence, because of the high number of analyses in the current work, specifying only one module appeared

to provide reasonable accuracy in the seismic analyses of RC buildings. The modeling approach is explained in detail next.

Geometry and Configuration of the Model

A schematic of the numerical model of the wall is shown in Fig. 1(a), including the configuration of nodes and elements. The discretized fiber section of the core wall is shown in the Fig. 1(b).

General WCM Model

Each C-shaped section of each core wall was separated into three rectangular regions (i.e., two flanges and the web), modeled by vertical elements in their centroid representing each segment. A horizontal link was used along the length of each segment to connect these vertical elements. The corner areas of the core section were divided in two and assigned to the web and flange sections. Hence, the bars located in the inner corners of the section were assigned two times (web and flange sections) to represent the outer bars also.

Beam-column elements with displacement-based formulation were used for piers in the model by assuming a constant axial strain and a linear distribution for the curvature. Both the axial elongation and the combined axial load and bending moment interaction of the RC sections could be taken into account using these elements. These effects are of the utmost importance in simulation of C-shaped RC cores in which variation in axial load in elements is expected when subjected to a bidirectional loading protocol.

Each element was set to have five Gauss integration points along its length. Full base fixity was assumed for the walls, and no soil-structure interaction was considered based on the results of a study by NIST (2010). However, slippage in the wall-footing intersections was considered (see the next section). The number of concrete and reinforcement fibers was different for the models, but was selected to create fine discretization in the section and to provide desirable accuracy. The confinement effect of the concrete materials was considered based on the model proposed by Mander et al. (1988).

As for the displacement-based elements, proper selection of the element length controls undesirable strain localization, in which

plastic deformations start to occur in the first layer of elements at the base of the wall while the elements in the upper layers are still at the elastic stage (Calabrese et al. 2010). Stafford-Smith and Girgis (1986) suggested limiting the spacing between horizontal links to approximately 20% of the overall height of the wall. Consequently, in the verification models in the current work, the height of each wall specimen was divided into 4 equal-length parts. Keeping approximately the same aspect ratio, the number of divisions was selected to be equal to 2 in each story (i.e., 2 rigid links) in simulation of the 12-story building.

As for the number of elements between links, Beyer et al. (2008b) found that a WCM with two elements between successive links suits most engineering studies. Therefore, extra nodes between successive links were introduced to equally divide the available height into two elements [Fig. 1(a)].

The horizontal links should have common nodes at the corners and be modeled as rigid links, except for torsional flexibility (Beyer et al. 2008b; Reynouard and Fardis 1993). The torsional flexibility (GK_t) assigned to the horizontal links was calculated using Eq. (1), as suggested by Xenidis et al. (1993):

$$GK_t = G \frac{h_l t_w}{3} \quad (1)$$

where G = shear modulus of concrete; K_t = torsional stiffness; h_l = rigid link spacing; and t_w = thickness of wall.

Validation of the results against available hysteresis results from experimental tests was conducted to examine the adequacy of the number of elements in the model. The selected number of elements was found to be sufficient for predicting the cyclic response of C-shaped walls, as explained in subsequent sections.

Because of the lack of shear flexibility in the selected type of elements, the two elements defined between successive rigid links were connected by zero-length spring elements to consider the shear flexibility of the wall segments. Hence, for both the in-plane and out-of-plane directions, a horizontal translational degree of freedom (DOF) in the zero-length element was assigned to have stiffness corresponding to the shear stiffness of the two wall elements connected to the spring element. *OpenSEES* EqualDOF type constraints were also used to couple the other DOFs of the two nodes. Unlike the moment capacity of walls, estimation of shear capacity of even a rectangular wall comes with large differences between available code provisions or shear models published in the literature. Beyer et al. (2008b) suggested that, for both the flanges and the web of the RC core wall, shear behavior can be considered elastic behavior using the uncracked section shear stiffness (k_s) computed by Eq. (2):

$$k_s = \frac{GA_s}{h_l} \quad (2)$$

where A_s = 80% of the gross section area of each segment (i.e., flanges and web).

This assumption has a considerable lack of accuracy once a shear mechanism in the wall segment occurs. In this study, a bilinear trend was used instead to consider the shear failure in the wall segment response. Shear strain at yield was set to be equal to 0.0015 based on the study by Massone et al. (2006).

P-δ Effects

Lateral displacement of a building caused by the seismic (and wind) loads on the lateral resisting system of the structure is usually accompanied by further displacements because of the P - δ effect. In the simulation of the 12-story building (Fig. 2), the effects of geometrical nonlinearity were also considered in the model by adding

a P - δ frame (leaning column analogy). This P - δ frame was located at a distance equal to the typical span length of the building (6 m) in the east-west direction [Fig. 2(c)]. The frame consisted of rigid corotational beam-column elements (columns) and horizontal rigid links connecting the columns to the rigid links at the corner of the section web at each story level. Because of the 3D modeling approach used in the study, P - δ frame elements were also connected to the core through diagonal truss elements to avoid instability of the system. The floor mass was computed at the story levels and was lumped at the corresponding nodal point. The tributary area considered for the P - δ frame at each story level was set as the total floor (or roof) area, from which the tributary area of the core wall system was excluded. A significant effect was observed on the lateral response of the building due to the P - δ effect, especially for higher lateral displacements.

FRP Elements

Effects of horizontal FRP wrapping on the behavior of wall segments were considered through confining the concrete materials and increasing the stiffness of horizontal springs in a way that represented the shear stiffness of FRP wraps and the wall section itself. Guidelines proposed by ACI 440 (ACI 2008) for FRP strengthening of reinforced concrete shear walls were considered at this step. Vertical FRP strips and FRP X-bracing were also modeled using uniaxial “elastic-no tension” material object in *OpenSEES*. Two options were available for implementing the vertical FRP strips: (1) fiber-based definition of FRP layers into the section of each wall segment, similar to the scheme proposed by ACI 440 [Figs. 3(c and b)] using separate spring elements connecting the corresponding nodes on the rigid links. To obtain comparable results with the wall strengthened using X-braced FRP strips, the second option was used in the current study. The elements were defined by connecting the corner and middle nodes and the nodes created at the end of flanges to define the coupling beams in the model [Fig. 3(b)]. Moreover, vertical FRP wrapping of the coupling beams was considered for all of the FRP-strengthened models. A schematic of the FRP retrofitting layout is shown in Fig. 4.

Perfect anchoring of the FRP strips was assumed at story levels. Moreover, to create a debonding rule at the midstory height, the nodes on the rigid link were duplicated and the bond-slip relationship model proposed by Lu et al. (2005) was implemented in the model. Tributary areas of FRP strips were calculated, and slip based relationships were defined to couple the nodes in the model. The other DOFs were slaved using the EqualDOF constraints in *OpenSEES*.

Strain Penetration Effects

In addition to the flexural/shear deformations in RC members, the model had to capture end rotation due to reinforcement slip, which occurs not only at the large inelastic stage but also during the elastic response of members (Huang 2012). This effect, called the strain penetration effect, was shown by experimental observations (Kowalsky et al. 1999) to make a relatively considerable contribution to the total lateral deformation of flexural members. To avoid overestimating the stiffness of the RC core, this effect should not be ignored in the modeling of RC cores because doing so can lead to underestimating the lateral drift of the core wall. Because the fiber-based modeling approach corresponds to the basic assumption that plane sections remain plane (i.e., perfect bond condition), the strain penetration effect can be considered by using a zero-length element at the base of the wall segments. Fig. 5 shows the schematic location of a zero-length element at the base of the fiber-based model of the RC core.

In the current work, the stress-slip model proposed by Zhao and Sritharan (2007) was used to take into account the strain

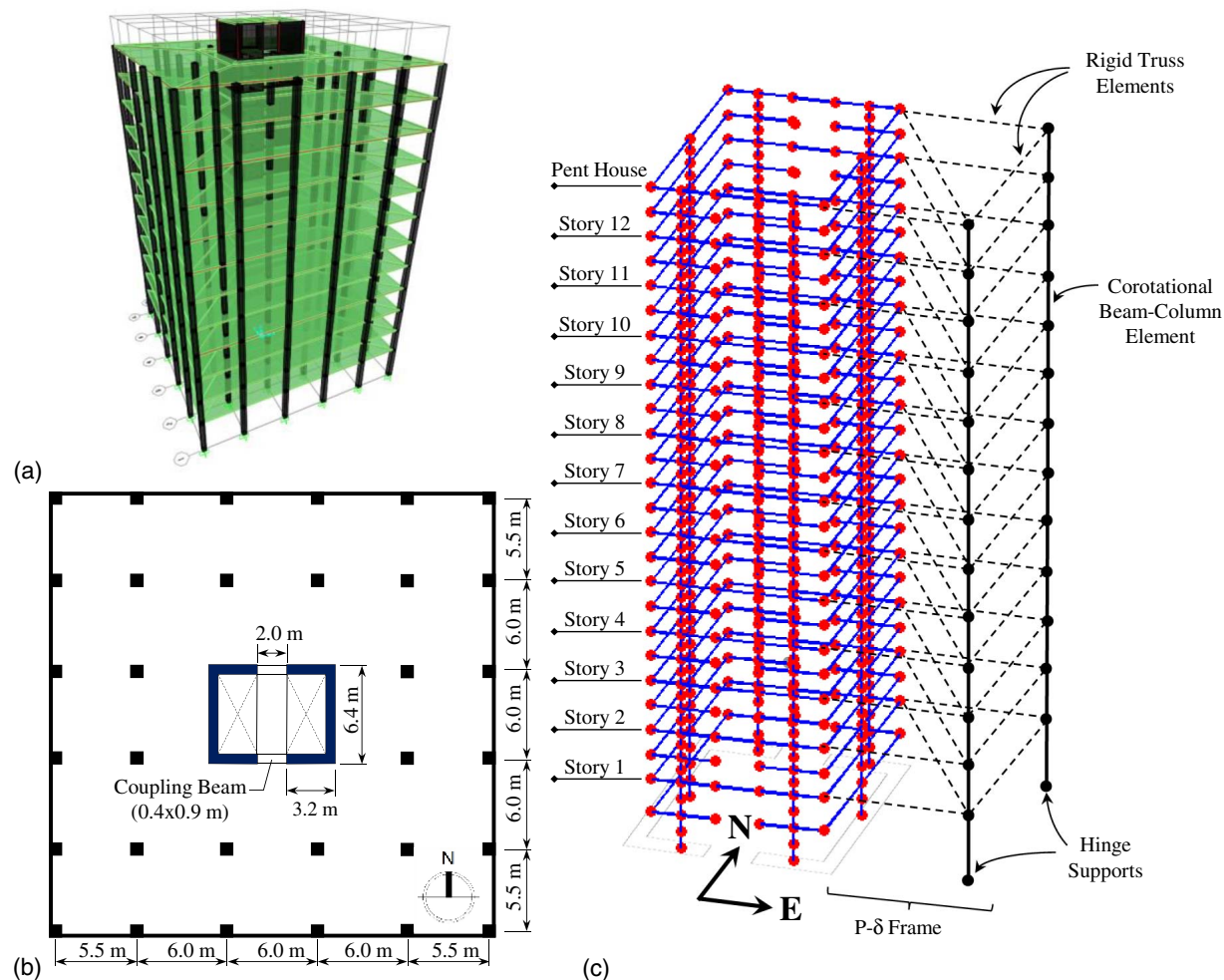


Fig. 2. (a and b) 3D layout and plan view of 12-story building coupled C-shaped RC core; (c) wide column model for the 12-story core wall

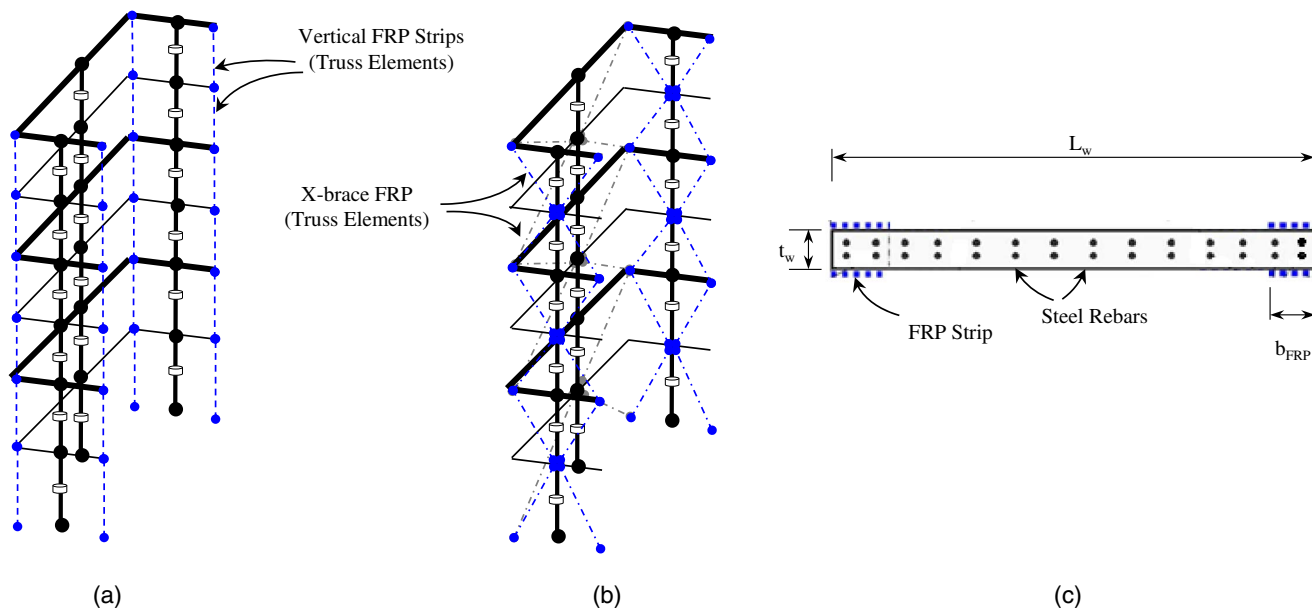


Fig. 3. Configuration of FRP elements in the model: (a) vertical FRP strips; (b) X-brace FRP strips; (c) typical cross section of core wall segments with vertical FRP strips proposed by ACI 440-2R (ACI 2008)

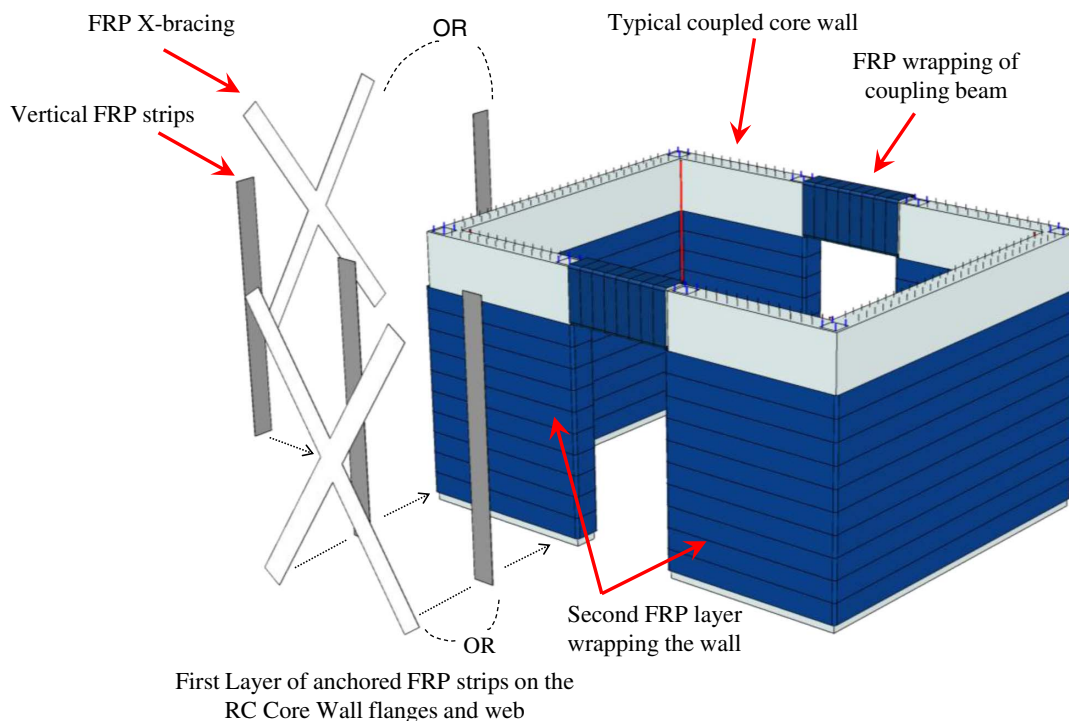


Fig. 4. Schematic of the FRP retrofitting layout

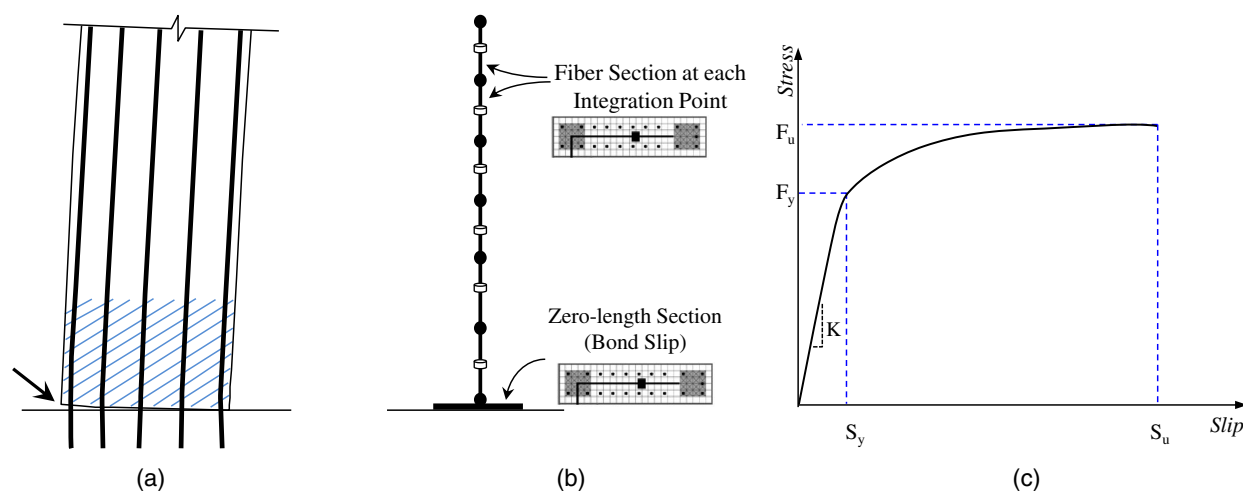


Fig. 5. Bond-slip components used to consider strain penetration effects: (a) schematic; (b) fiber-based model for each core wall segment; (c) stress-displacement relationship developed by Zhao and Sritharan (2007)

penetration effects in wall-footing intersections. *OpenSEES* Bond_SP01 material was used to represent the vertical reinforcements. This model considers the total bar slip caused by strain penetration as a function of a certain level of stress in the bar (Zhao and Sritharan 2007).

Failure Criteria for Materials

Mechanical properties of materials were defined based on the reported test data or the nominal strength of concrete and steel reinforcement using the available empirical equations. However, confinement effects on concrete behavior were also taken into account. Fig. 1(b) shows the distribution of different materials

in the cross section of the RC cores simulated in the numerical model.

A modified Kent and Park model (Scott et al. 1982) was used to define the nonlinear constitutive laws for concrete fibers (Concrete02 in *OpenSEES*). Though the model proposed by Chang and Mander (1994) has a few more accurate features such as better simulation of gradual closure of cracks under cyclic loading and calibration of the stress-strain data resulting from experimental tests (Concrete07 in *OpenSEES*), the Concrete02 model was used because of its simplicity of calibration. The model offers a reasonable level of accuracy while holding its simplicity and is extensively used even though some later constitutive models have proved to be more precise and comprehensive

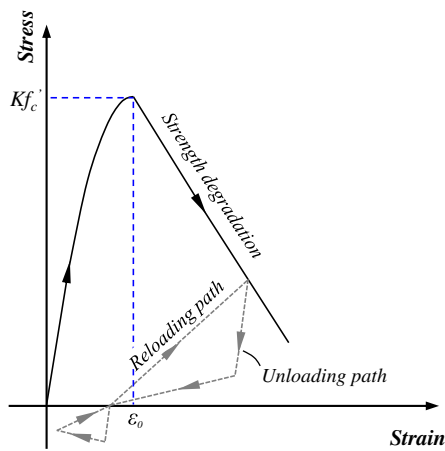


Fig. 6. Hysteretic unloading and reloading rules used in the model (reprinted from E. Spacone, F. C. Filippou, and F. Taucer, "Fibre Beam-Column Model for Non-Linear Analysis of R/C Frames: Part I. Formulation," *Earthquake Engineering and Structural Dynamics*, Copyright © 1998 John Wiley & Sons, Ltd., with permission from John Wiley and Sons)

(Orakcal et al. 2006). The monotonic compressive stress-strain ($\sigma_c - \varepsilon_c$) curve can be described by three regions as follows (Fig. 6):

$$\sigma_c = Kf'_c \left[2 \left(\frac{\varepsilon_c}{\varepsilon_0} \right) - \left(\frac{\varepsilon_c}{\varepsilon_0} \right)^2 \right]; \quad \varepsilon_c \leq \varepsilon_0 \quad (3)$$

$$\sigma_c = Kf'_c \left[2 \left(\frac{\varepsilon_c}{\varepsilon_0} \right) - \left(\frac{\varepsilon_c}{\varepsilon_0} \right)^2 \right]; \quad \varepsilon_c \leq \varepsilon_0 \quad (4)$$

$$\sigma_c = 0.2Kf'_c; \quad \varepsilon_c > \varepsilon_{20} \quad (5)$$

The initial tangent moduli (E_c) can be calculated by the following expressions:

$$E_c = 4500\sqrt{f'_c} \text{ (MPa)} \quad (6)$$

where ε_0 = strain corresponding to concrete peak stress under compression; ε_{20} = strain corresponding to 20% of maximum compressive stress; K = confinement modification factor; f'_c = concrete compressive strength (unconfined peak stress in cylinder test, MPa); and Z = slope of strain softening.

A set of linear stress-strain relations was subsequently proposed by Yassin (1994) to define the hysteretic unloading and reloading rules for material models (Fig. 6). This model has been used by some researchers such as Spacone et al. (1996). The tensile strength of concrete (f'_t) was calculated as follows:

$$f'_t = 0.623\sqrt{f'_c} \quad (7)$$

where f'_t and f'_c are expressed in MPa.

In the current work, a similar form of the stress-strain curve was used for both unconfined and confined concrete. Peak compressive strength for the confined concrete was calculated based on the method proposed by Mander et al. (1988) for different regions of the cross sections, which are shown in Fig. 1(b).

For steel reinforcement, the Giuffr -Menegetto-Pinto hysteretic model (Filippou et al. 1983) with default parameters (Steel02 in *OpenSEES*) was adopted to represent the nonlinear transition from elastic stage to strain hardening stage for cyclic and pushover analyses. This model defines steel material objects with isotropic strain hardening. Input data to define this relationship are elastic modulus E_s , yield strength f_y , and the coefficient that defines the relationship between the elastic modulus and the slope of tangent after reaching the yield point. In cases where the ultimate strength of the steel bars is not known (e.g., the studied 12-story building in the current work), the hardening ratio equal to 0.5% can be assumed to be suitable for most 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 12-story building. Sample plots of the cyclic material behavior employed for the steel and concrete fibers in the numerical model of Specimen TUA are depicted in Fig. 7. Details and mechanical properties of different FRP layers are summarized in Table 2. The same FRP materials were used in all retrofitting schemes.

Model Validation

In order to validate the modeling approach and the assumed failure criteria, two RC wall specimens from different experimental programs available in the literature were selected to be modeled using the approach just explained. The results of the numerical simulations were compared with the reported experimental results. The lateral force acting on the wall was normalized to the maximum shear resistance allowed by ACI 318-14 (ACI 2014) ($V_n = 0.666A_{cv}\sqrt{f'_c}$).

Core Walls Tested by Beyer et al.

Beyer et al. (2008b) performed four tests on C-shaped shear walls with different wall thickness and reinforcement configurations to evaluate the performance of this type of wall geometry when subjected to bidirectional loading protocols. Specimen TUA was a C-shaped RC core 2,720 mm in height, 150 mm in wall thickness, 1,300 mm in width, and 1,050 mm in depth. The top of the core was attached to a concrete collar 300 mm in thickness, providing joints to be attached to three actuators (two acting on the flanges and one on the web). As noted in Table 1, D6 and D12 bars were used for

Table 1. Sample Material Parameters for the *OpenSEES* Model

Specimen	Region	Concrete material						Steel material		
		f'_c	e_0	K	f_{cu}	e_u	f_t	Rebar	f_y	f_u
TUA (Beyer et al. 2008a)	Unconfined region	-77.9	0.0037	1	-15.58	-0.010	5.50	D12 mm bars	488	595
	Confined region 1	-87.3	0.0039	1.121	-16.70	-0.010	5.69	D6 mm bars	518	681
	Confined region 2	-97.8	0.0042	1.256	-19.57	-0.010	6.16	—	—	—
Wall8 (Lowes et al. 2013)	Unconfined region	-35.2	0.0025	1	-7.03	-0.010	3.69	#2	531	592
	Confined region 1	-41.5	0.0027	1.18	-8.30	-0.010	4.01	#4	441	630
12-story building	Unconfined region	-30.0	0.0023	1	-6.00	-0.010	3.41	10M	400	545
	Confined region 1	-31.2	0.0024	1.0404	-6.24	-0.010	3.48	25M	400	545

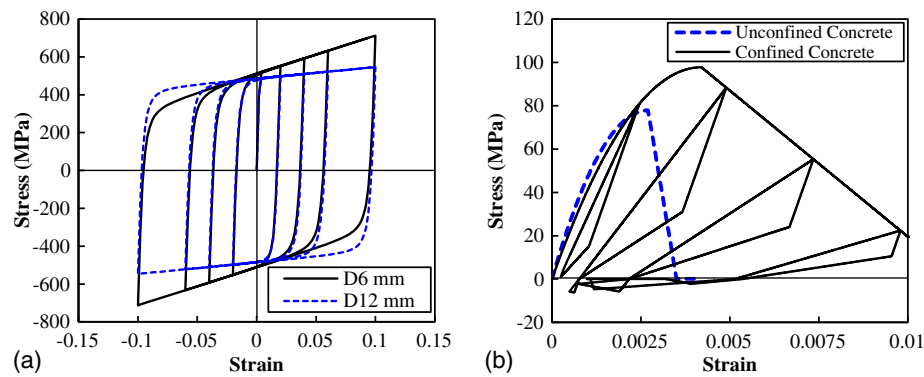


Fig. 7. Sample hysteresis stress-strain behavior of material assigned in the numerical model: (a) reinforcement; (b) concrete

Table 2. Details and Mechanical Properties of Different FRP Layers

FRP layer	Type	Thickness (mm)	Strip width (mm)	Number of layers	f_u (MPa)	E_{FRP} (MPa)	$\epsilon_{rupture}$ (%)
Horizontal wraps	CFRP	0.2	—	2	1,062	102,000	1.05
Vertical FRP strips	CFRP	0.2	800	1	1,062	102,000	1.05
FRP X-braces	CFRP	0.2	1,200	1	1,062	102,000	1.05
Vertical wraps (coupling beams)	CFRP	0.2	—	2	1,062	102,000	1.05

steel reinforcement in the flanges/web and corner boundary zones, respectively. A high-strength concrete material ($f'_c = 77.9$ MPa) was used for constructing the core. The core was subjected to an axial force equal to 780 kN (core wall self-weight plus the loads applied by experimental equipment). This axial force was kept constant during the cyclic test, and a bidirectional loading protocol with consecutive cycles corresponding to different levels of ductility was applied to the core.

Fig. 8(a) shows the normalized load-drift data at the top of the wall from the experimental test along with corresponding results from the numerical model. The figure shows good agreement between the experimental records and the numerical predictions with small discrepancies. The small discrepancies between the test results and numerical predictions, acceptable for such a simplified model, might be due to the shear flexibility assigned to the rigid links, which violate the compatibility of strains between the flange and the web section at the corner of the core.

Core Walls Tested by Lowes et al.

The experimental work of Lowes et al. (2013) consisted of testing three C-shaped RC core specimens with similar designs representing the bottom three stories of a modern midrise building. The objective of their study was to investigate the impact of bidirectional loading on the earthquake performance of isolated C-shaped walls and C-shaped walls in coupled core wall systems. Specimen Wall8 was a C-shaped RC core 3,660 mm in height, 152.4 mm in wall thickness, 3,048 mm in width, and 1,219.2 mm in depth. A wall cap 457.2 mm in height was constructed to apply a cruciform lateral load pattern to the wall specimen. The axial load was maintained constant (equal to 5% of the gross axial capacity: $0.5f'_c A_g = 1,361$ kN) when the wall was subjected to lateral loading in the strong direction. As can be seen in Fig. 8(b), there is a very good correlation between the normalized lateral load-drift curves from the numerical analysis and those from the experimental work of Lowes et al. (2013). As the displacement increases, however,

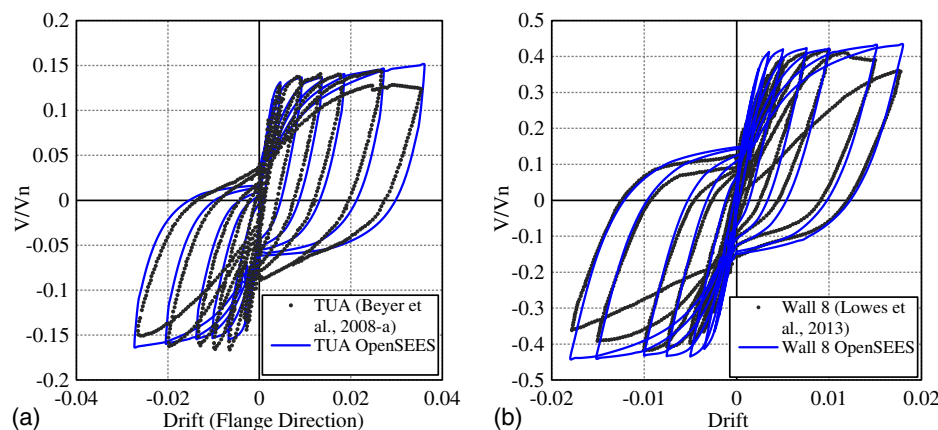


Fig. 8. Comparison of results from the numerical model and the experimental data: (a) Specimen TUA tested by Beyer et al. (2008a); (b) Specimen Wall8 tested by Lowes et al. (2013)

Table 3. Summary of Verification Results: Numerical Predictions in Comparison with Experimental Data

Specimen	Numerical/experimental ratio				
	$V_{\max}$	$K_{\text{initial}}^{\text{loading}}$	$K_{\text{unloading}}^{\text{unloading}} @ 1\% \text{ drift}$	Δ_y	V_y
TUA (Beyer et al. 2008a)	0.98	0.97	1.02	0.92	1.06
Wall 8 (Lowes et al. 2013)	1.06	1.10	1.07	0.94	1.10

the numerical model shows a bit higher load resistance than the experimental model.

Comparisons between the main output results from the numerical models and the experimental work showed satisfactory accuracy of the model in tracking the lateral response of C-shaped RC cores. Peak loads in reversal cycles, along with general load-displacement trends, well matched experimental observations. Table 3 summarizes the results from the verification models, including the maximum shear capacity of RC cores, $V_{\max}$; lateral rigidity, $K_{\text{initial}}^{\text{loading}}$; unloading stiffness in the cycle corresponding to approximately 1% drift, $K_{\text{unloading}}^{\text{unloading}} @ 1\% \text{ drift}$; yield displacement, Δ_y ; and applied shear corresponding to the yield point, V_y . From the comparisons, it can be seen that the average error percentage for each set of the compared results shows that both the failure criteria and modeling approach used for the numerical analysis were acceptable.

The validated model was then used to perform a seismic collapse risk assessment on a typical 12-story building with different levels of torsional sensitivity.

IDA Analysis of a Typical 12-Story Building

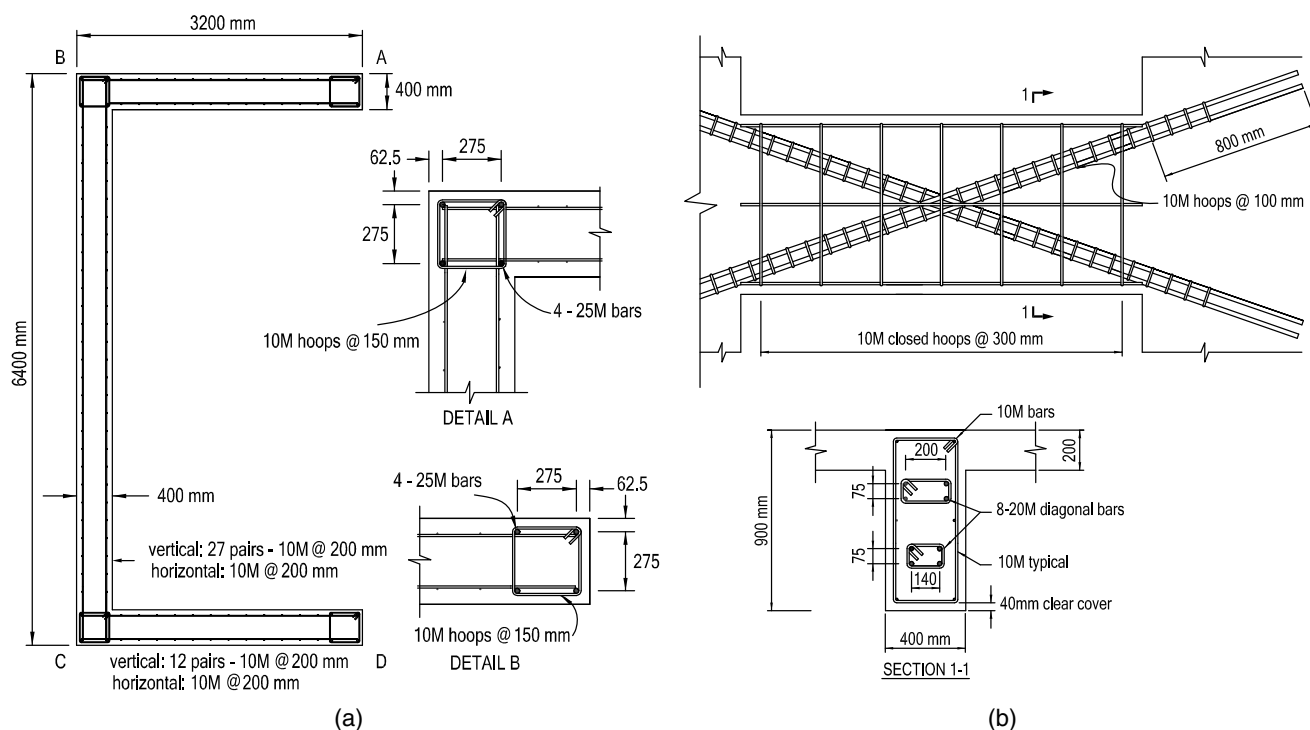
A 12-story RC building in Montreal, Canada, with Class-D subsoil conditions was selected for seismic collapse risk assessment using the FEMA P695 methodology. The initial building design followed

the Cement Association of Canada (CAC) *Concrete Design Handbook* (CAC 2010), the National Building Code of Canada (NBCC 2005), and the CSA A23.3-04 standard (CSA 2004).

Each floor consisted of 5.5-m end spans and three 6-m interior spans in both horizontal directions, resulting in an outside-to-outside plan dimension of 29.75 m. Considering the penthouse on the roof, the total height was 48.65 m, consisting of a 4.85-m-high first story and a typical story height of 3.65 m. The 3D view and the plan of the building are shown in Fig. 2. The gravity system consisted of 200-mm-thick flat plate slabs supported by 550-mm square columns. The core wall measured 6.4×8.4 m, outside to outside of the walls and consisted of two C-shaped RC cores 400 mm in thickness connected through 900-mm-deep coupling beams, which were centrally positioned in the plan of the building. The reinforcement layout at the end of the flanges and at the web-flange junctions was considered to be 4–25-M bars, whereas the distributed reinforcement was considered to be 10-M vertical and horizontal bars with 200 mm spacing. The reinforcement for coupling beams was considered to be 8–20-M diagonal bars. Geometrical details of the coupled RC core wall are shown in Fig. 9. The building was designed to be built using 30-MPa-strength concrete and 400-MPa-strength steel reinforcement. The structural damping was considered by means of mass and stiffness proportional Rayleigh damping. A damping ratio of 5%, a typical value for RC buildings, was assigned to the first two modes of the structure. Although the Rayleigh damping feature in *OpenSEES* can formulate the damping matrix using the initial, current, or last committed stiffness matrix, only the initial stiffness matrix was used in the current work. However, the rigid truss elements that linked the frame and the leaning columns were excluded.

Torsional Sensitivity

Both Canadian and American design codes have limits in classifying the “torsional sensitivity” of building structures. Torsional

**Fig. 9.** Geometrical and reinforcement configuration of the coupled RC core wall: (a) C-shaped walls; (b) coupling beams (CSA A23.3-04)

sensitivity in a building is defined as the maximum value B among floors, where B_x computed at Level x is $B_x = \delta_{\max}/\delta_{\text{ave}}$. In the aforementioned expression, $\delta_{\max}$ is the maximum story displacement, calculated at Level x considering 10% accidental torsion, at one end of the structure, and δ_{ave} is the corresponding average displacement for the two end points of the structure. According to NBCC (2015), when the value of B exceeds 1.7 and $I_E F_a S_a(0.2) > 0.35$, the building is considered torsionally sensitive. In this case, results from static analysis are no longer adequate, and a 3D dynamic analysis is required for evaluating the response of the structure. In the current work, because the building selected for the collapse assessment was designed according to Canadian standards, the NBCC (2015) limit was used to classify torsional irregularities. The IDA results first collected for three different levels of torsional sensitivity ($B = 1.0, 1.7$, and 2.5). The torsional flexibility of the building was increased by providing an inherent mass eccentricity e_x through artificial shifting of the center of mass (CM) from the center of rigidity (CR). This amount of e_x was obtained based on an iterative procedure by analyzing the structure against the design-level earthquake. Moreover, the accidental torsion equal to 5% was taken into account. Subsequently, the model with $B = 1.7$ was analyzed with different FRP strengthening schemes, including horizontal FRP wrapping, using vertical FRP strips, and X-bracing with FRP strips.

Ground Motion Selection

According to FEMA P58-1 (FEMA 2012), “a minimum of 7 ground motion records is recommended because of extremely poor prediction of record-to-record variability obtained in small suite of ground motions” (quoted in Michaud and Léger 2014), regardless of the spectral matching efficiency. Moreover, using more than 44 ground motion records for seismic collapse risk assessment of buildings is recommended by FEMA P695 to achieve more reliable results. However, high computational effort is always a drawback. Because the considered building was located on Site Class D in Montreal, and because of the lack of real seismic wave data from the past in Canada, in the current work 50 spectrum-compatible artificial accelerograms were used for IDA analysis of the building. These records were divided into two groups: (1) “Near-Fault”—ground motions of magnitudes (M) between 5.5 and 6.5 with the closest distance to the fault, R_{fault} , from 0 to 15 km; (2) “Far-Field”—ground motions of magnitudes (M) between 6.75 and 7.25 with R_{fault} from 10 to 90 km. These accelerograms were developed by Atkinson (2009) for design of structures in Montreal, and their characteristics, including magnitude, R_{fault} , peak ground acceleration (PGA), and max velocity to max acceleration ratio (v/a) are summarized in Tables 4 and 5. As shown, records in Group 1 produce a higher v/a , which confirms the expected response of structures under near-fault excitations.

There are no specific provisions in NBCC (2015) regarding scaling of ground motions. However, it is indicated that scale factors should be applied for all selected ground motions to match the uniform hazard spectrum (UHS) at the first mode period of the structure. Moreover, scaled ground motions should match or have larger intensity than the UHS at all points representing the period of higher modes. On the other hand, ASCE/SEI-7 (ASCE 2010) states that the mean of the 5% damped response spectra for a group of seven or more ground motions should fit or be above the UHS in all the points within the range of $0.2T_1$ – $1.5T_1$.

Considering the ASCE/SEI 7-10 (ASCE 2010) standard requirements, all selected records were scaled to reach a mean

Table 4. Unscaled Selected Near-Fault Ground Motions

Event	M	R_{fault} (km)	PGA (g)	v/a
east6d1.1	6	12.8	0.707	0.0663
east6d1.2	6	12.8	0.523	0.0496
east6d1.3	6	12.8	0.591	0.0276
east6d1.4	6	12.5	0.671	0.0559
east6d1.5	6	12.5	0.471	0.0448
east6d1.6	6	12.5	0.678	0.0415
east6d1.7	6	12.8	0.491	0.0686
east6d1.8	6	12.8	0.593	0.0443
east6d1.9	6	12.8	0.370	0.0413
east6d1.10	6	12.8	0.366	0.0483
east6d1.11	6	12.8	0.410	0.0502
east6d1.12	6	12.8	0.515	0.0404
east6d1.13	6	10.7	0.570	0.0419
east6d1.14	6	10.7	0.568	0.0362
east6d1.15	6	10.7	0.941	0.0318
east6d1.16	6	13.6	0.606	0.0596
east6d1.17	6	13.6	0.461	0.0416
east6d1.18	6	13.6	0.609	0.0540
east6d1.19	6	14.4	0.510	0.0483
east6d1.20	6	14.4	0.433	0.0798
east6d1.21	6	14.4	0.484	0.0491
east6d1.22	6	14.4	0.452	0.0357
east6d1.23	6	14.4	0.557	0.0333
east6d1.24	6	14.4	0.649	0.0507
east6d1.25	6	14.4	0.535	0.0448
Average	6	13.1	0.550	0.0474

Table 5. Unscaled Selected Far-Field Ground Motions

Event	M	R_{fault} (km)	PGA (g)	v/a
east7d1.1	7	13.8	0.627	0.0802
east7d1.2	7	13.8	0.623	0.1150
east7d1.3	7	13.8	0.764	0.0594
east7d1.4	7	15.3	0.767	0.0832
east7d1.5	7	15.3	1.183	0.0781
east7d1.6	7	15.3	0.866	0.0533
east7d1.7	7	14.2	0.840	0.0909
east7d1.8	7	14.2	0.813	0.0706
east7d1.9	7	14.2	1.152	0.0919
east7d1.10	7	14.9	0.968	0.0795
east7d1.11	7	14.9	0.997	0.0522
east7d1.12	7	14.9	0.902	0.1139
east7d1.13	7	14.8	0.950	0.0702
east7d1.14	7	14.8	0.944	0.0714
east7d1.15	7	14.8	0.928	0.0935
east7d1.16	7	20.6	0.635	0.0833
east7d1.17	7	20.6	0.606	0.0496
east7d1.18	7	20.6	0.403	0.1003
east7d1.19	7	20.1	0.668	0.0817
east7d1.20	7	20.1	0.487	0.0609
east7d1.21	7	20.1	0.655	0.0754
east7d1.22	7	14.3	0.960	0.0732
east7d1.23	7	14.3	0.768	0.0853
east7d1.24	7	14.3	1.101	0.0787
east7d1.25	7	19.6	0.538	0.0804
Average	7	16.1	0.806	0.0789

spectrum that matched the design spectrum over the interval of $0.2T_1$ – $1.5T_1$, where T_1 is the first mode period of vibration obtained by conducting a modal analysis in *OpenSEES*. Fig. 10 shows the response spectra of the scaled ground motions together with the mean spectrum and the design spectrum proposed by NBCC for Montreal.

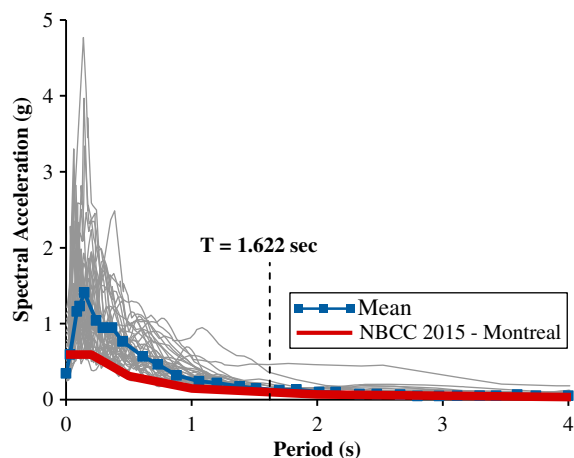


Fig. 10. Response spectra for 50 scaled artificial ground motion records for Montreal

Nonlinear Structural Analysis

Building Configurations

Dynamic time history analyses were performed to evaluate the collapse capacity of the building in the east-west direction, the direction in which the RC core wall acted as coupled wall system. As mentioned, the 12-story RC core system was analyzed in six different configurations as follows:

- Original structure with torsional sensitivity value $B \approx 1.0$;
- Original structure with torsional sensitivity value $B = 1.7$;
- Original structure with torsional sensitivity value $B = 2.5$;
- Structure with $B = 1.7$ and strengthened with horizontal FRP wrapping (H-FRP);
- Structure with $B = 1.7$ and strengthened with horizontal wrapping and vertical FRP strips (H&V-FRP); and
- Structure with $B = 1.7$ and strengthened with horizontal wrapping and X-brace FRP strips (H&X-FRP).

Each structure was analyzed for 50 ground motion records and with at least 16 different intensity factors. Hence, 2,400 time history analyses were conducted in total. The number of analysis steps and the time step of the analysis should be specified properly to execute the dynamic time history analysis. It must be less than or equal to the sampling rate of the input ground motion. In the current work, the time step of each analysis was specified as $\Delta t = 0.01$ s, which was half of the sampling rate of the input ground motions.

Analytical Failure Modes

Despite the several failure criteria adapted for the material behavior in the model, some failure modes could not be directly simulated in the numerical model because of limitations in *OpenSEES*. These failure modes had to be assessed through postprocessing of the recorded data. Considering the criteria proposed by Gogus and Wallace (2015), the following failure modes were tracked in the model outputs:

- Steel buckling/fracturing: although the MinMax feature in *OpenSEES* was used to limit the strain capacity of steel bars, failure of a single row of rebars (wall thickness direction) was found to be insufficient to cause collapse (Thomsen and Wallace 1995); thus, buckling or fracture of the reinforcement positioned at approximately one-quarter length of each segment of the core wall was considered as the collapse point [Fig. 11(a)];

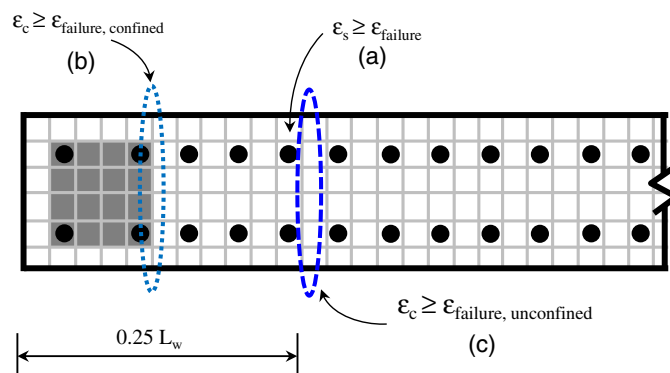


Fig. 11. Reinforcing steel and concrete failure mode criteria: (a) steel rebar failure; (b) concrete crushing in confined region; (c) concrete crushing in unconfined region

- Concrete crushing: similar criteria established for concrete crushing, in which the failure corresponds to reaching the crushing strain for concrete fibers positioned at approximately one-quarter length of the wall segment, in the unconfined region; as for the confined region, crushing strain at the innermost confined concrete fiber was tracked to find the crushing point [Figs. 11(b and c)];
- Axial failure: 5% drift limit at axial failure was selected for a 12-story archetype, based on the model developed by Wallace et al. (2008) for lightly reinforced wall piers and considering the details published by GCR 10-917-8 (NIST 2010); and
- Shear/flexural failure of the wall: by developing the cracks in the tension wall in a coupled shear wall system, penetration of the flexural cracks near the compression edge was expected; as a result, a greater contribution from the dowel action of the vertical reinforcement was expected for resisting the sliding shear failure; this increased the probability of shear sliding failure in the wall (Pauly and Priestly 1992), which was captured by applying the corresponding limits on the horizontal springs representing the shear behavior of the wall segments.

Failure of the coupling beams was not considered as a collapse mechanism for the system. In fact, the coupling beams act as a fuse in ductile coupled shear walls, dissipating most of the seismic energy input. However, as a second energy-absorbing line of defense, the walls should also be detailed to accommodate plastic hinging at the base without excessive loss of strength to avoid collapse after all coupling beams have yielded (Honarparast and Chaallal 2015).

Once a failure mode happens at a certain time in the analysis (t_{failure}), the estimated redistribution of forces after t_{failure} is no longer reliable. However, in the current work, t_{failure} was considered a failure moment and the results afterward were excluded from the analysis data. This postprocessing procedure helped to minimize the effects of the observed errors in capturing the postpeak strength degradation (i.e., discrepancies in the results of verification models) on the collapse assessment.

Results and Discussion

The impact of dynamic loads and higher modes on the performance of structures designed based on CSA A23.3-04 was investigated using incremental dynamic analyses (Vamvatsikos and Cornell 2002) before and after FRP retrofitting. IDA analyses were conducted on each archetype according to the FEMA

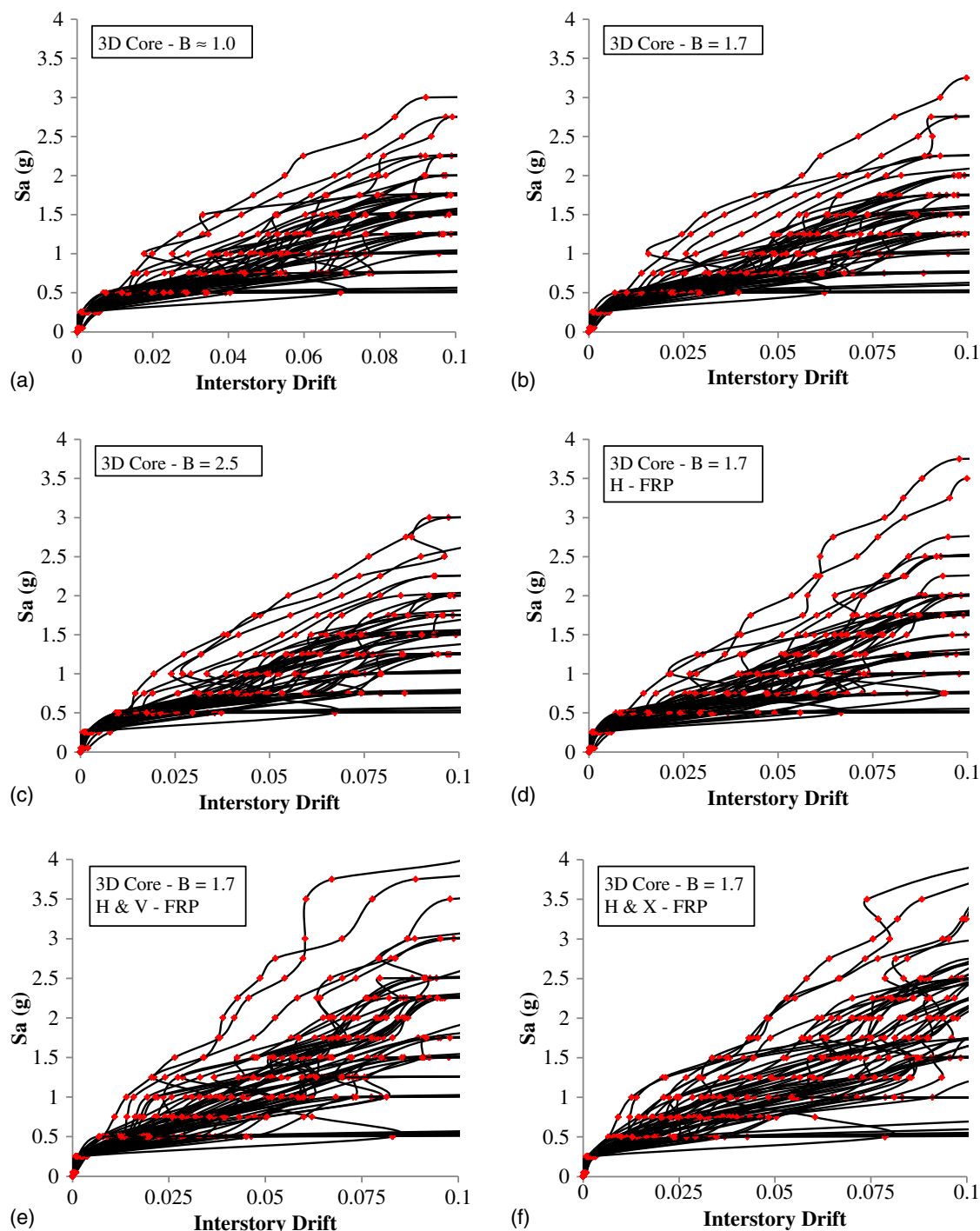


Fig. 12. IDA response plot of 12-story building: (a) $B \approx 1.0$; (b) $B = 1.7$; (c) $B = 2.5$; (d) $B = 1.7$ (strengthened with horizontal FRP wrapping); (e) $B = 1.7$ (strengthened with horizontal and vertical FRP); (f) $B = 1.7$ (strengthened with horizontal and X-brace FRP)

P695 methodology. After postprocessing the data using the aforementioned criteria, IDA response plots were created. Fig. 12 shows the IDA response plot of the structure in different configurations.

Collapse Fragility of the Structures

A collapse fragility function was developed for each case by cumulative distribution of the results obtained from the IDA analyses. The selected intensity measure (IM) in the current study was the 5% damped spectral acceleration at the fundamental period of the

structure [$Sa(T_1, 5\%)$]. The fragility curve represents the intensity of the ground motion versus the collapse probability (Ibarra et al. 2002). Although the scaling was performed for different IM levels in the IDA analysis, the fragility function-fitting method proposed by Baker (2015) was used to derive the fragility function.

The collapse probability of 50% for the structure, called the median collapse capacity (S_{CT}), and the collapse margin ratio (CMR) were calculated from the IDA results. Fig. 13 depicts a sample fragility curve obtained from the results of IDA analyses using the function-fitting method proposed by Baker (2015). The sample

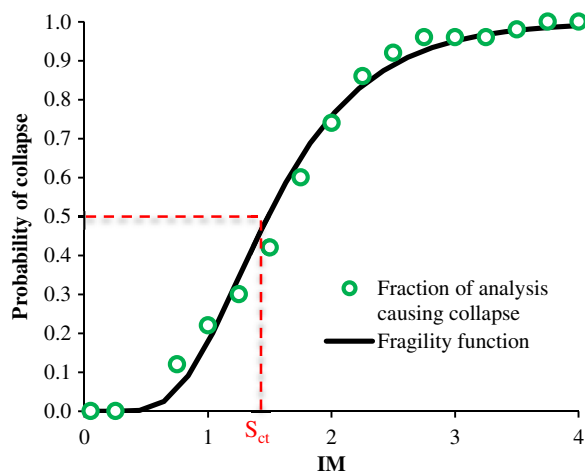


Fig. 13. Sample fragility function fitting for the results of the IDA

Table 6. Sample Data Set Used in the Development of the Sample Fragility Curve

IM	Number of analyses	Number of collapses	Fraction causing collapse	Theoretical fragility function
0.05	50	0	0.00	0.00
0.25	50	0	0.00	0.00
0.75	50	6	0.12	0.05
1	50	11	0.22	0.18
1.25	50	15	0.30	0.34
1.5	50	21	0.42	0.51
1.75	50	26	0.52	0.65
2	50	37	0.74	0.76
2.25	50	43	0.86	0.84
2.5	50	46	0.92	0.89
2.75	50	48	0.96	0.93
3	50	48	0.96	0.95
3.25	50	48	0.96	0.97
3.5	50	49	0.98	0.98
3.75	50	50	1.00	0.99

set of data used in the development of the fragility curve (Fig. 13) is summarized in Table 6, in which the term “Fraction Causing Collapse” is simply the ratio of the collapsed cases to the total number of analyses for each IM level. The term “Theoretical Fragility Function” corresponding to each IM level was calculated through a normal distribution for the specified median and standard deviation. The median and dispersion for the data set were calculated using the method proposed by Baker (2015).

The CMR, as a primary parameter for characterizing the collapse safety of structures, could be calculated as S_{CT}/S_{MT} , where S_{MT} is the maximum considered earthquake (MCE) intensity corresponding to the fundamental period, T , and S_{CT} is the median collapse intensity from the IDA results. It is worth mentioning that the fundamental period used in the FEMA P695 (FEMA 2009) methodology is the strength-based period, which is presented in Section 12.8 of ASCE/SEI 7-10 (ASCE 2010).

Fig. 14 compares the fragility curves obtained from IDA analyses of building structures. As can be seen from Fig. 14(a), the torsional sensitivity can dramatically affect the fragility function of the building in the IDA analyses. The median collapse capacity (S_{CT}) of the RC core system decreased approximately 8% by reaching the torsional sensitivity of $B = 1.7$. As for the structure with $B = 2.5$,

the reduction in the mean failure capacity was more than 36% in comparison with the structure with $B \approx 1.0$. The fragility function was also very steep for the structure with $B = 2.5$.

As for the strengthened RC cores, the structure strengthened with horizontal FRP wrapping showed an increase in collapse capacity up to 17% [Fig. 14(b)], whereas there was more significant improvement obtained by H&V-FRP and H&X-FRP configurations, which showed 44 and 63% increases, respectively [Figs. 14(c and d)]. Moreover, FRP-strengthened walls were found to obtain flatter fragility response in comparison with original walls. Table 7 summarizes the collapse assessment results of RC cores evaluated in the current study.

Fragility Curves at Different Performance Levels

Based on the results by Carrillo and Alcocer (2012), although the available drift limits for the performance levels defined by FEMA-356 (FEMA 2000) and ASCE41-13 (ASCE 2014) are not safe for nonductile structures, these acceptance levels can be conservatively used for ductile structures. However, damage states, after retrofitting, shift in terms of interstory drift ratio depending on the retrofitting method used. For the sake of comparison, in the current work damage states were kept constant corresponding to the performance levels proposed by ASCE41-13, and fragility curves for different configurations of the structure were developed accordingly. Immediate occupancy (IO), life safety (LS), and collapse prevention (CP) performance limits were considered for the seismic collapse assessment. Fig. 15 shows the results of collapse probability for different buildings. A comparison between the median collapse capacities is presented in Table 8 for different limit states. As can be seen, although the mean collapse capacities were similar for the original structures with different torsional sensitivities at the IO level, structures with higher torsional sensitivities resulted in 20–25% less failure capacity at the LS and CP levels. Close results were obtained for the retrofitted structures at the IO level, whereas up to 15 and 20% increases were observed for the failure capacities at the LS and CP levels, respectively. These drift-based results for the structures retrofitted with H-FRP and H&V-FRP were relatively close, although a significant enhancement was obtained by using H&X-FRP retrofitting.

Interstory Drift Ratio

Because interstory drift is an important indicator of structural behavior in performance-based seismic analysis, this section presents the variation in maximum interstory drift during the IDA analysis. For each building configuration, postprocessing was performed on the statistical data resulting from the IDA analysis. The results are shown in terms of box plots in Fig. 16, in which, for each story, the solid line shows the range of variation from the lower bond to the upper bond. The box represents the interquartile range (Q3–Q1), which starts from the lower quartile and finishes at the upper quartile, and the vertical solid line inside the box indicates the median. As shown, the lower bond and the upper bond for the maximum interstory drift ratios of the original structures were approximately 0.5 and 1.4%, respectively. Although the median of the drift data for each story fluctuates along this range, it peaks at 0.89, 0.82, and 0.79% for the structures with $B \approx 1.0$, $B = 1.7$, and $B = 2.5$ respectively. This shows a decrease of up to 12% in the peak median by increasing the torsional sensitivity of the structure. Although the torsional irregularity had effects in terms of torsion on the structural system, however, the interstory drift components of the system were not influenced significantly. This is because the

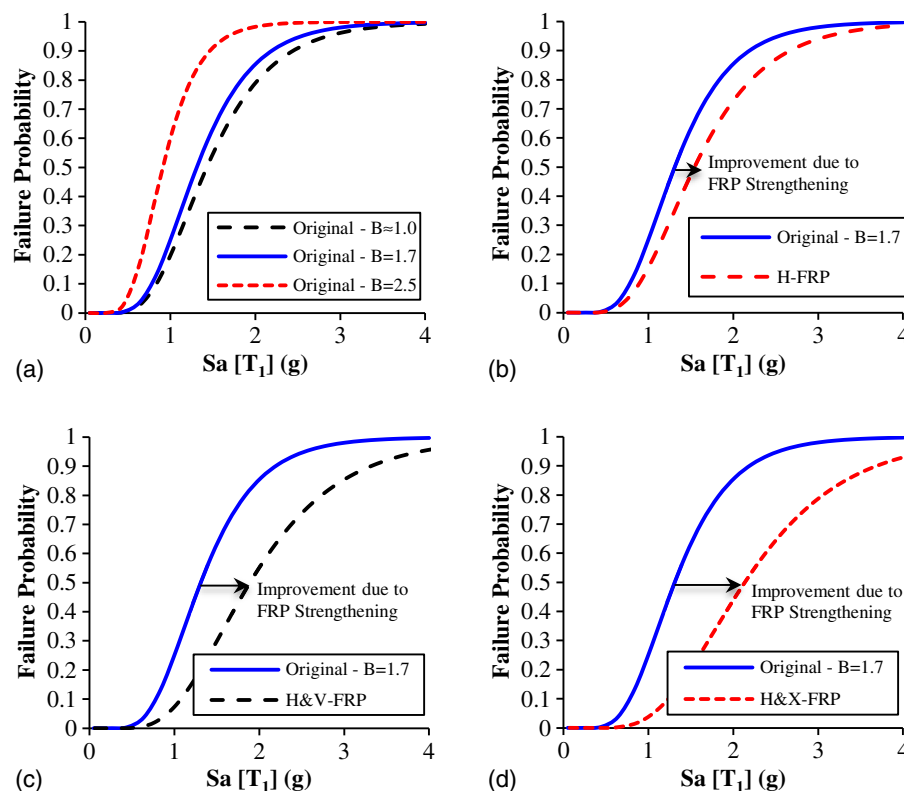


Fig. 14. Fragility curves of original and strengthened buildings: (a) original RC core with different torsional sensitivity; (b) strengthening with horizontal FRP wrapping; (c) strengthening with horizontal FRP wrapping and vertical FRP strips; (d) strengthening with horizontal FRP wrapping and X-brace FRP strips

Table 7. Summary of Collapse Assessment Results for RC Cores

Archetype identifier	Configuration	Strengthening scheme	B	S_{MT} [T] (g)	S_{CT} [T] (g)	CMR
O1	Original	—	$B \approx 1.0$	0.174	0.910	5.2
O2	Original	—	$B = 1.7$	0.174	1.310	7.5
O3	Original	—	$B = 2.5$	0.174	1.429	8.2
S1	Strengthened	Horizontal FRP	$B = 1.7$	0.174	1.544	8.9
S2	Strengthened	Horizontal and vertical FRP	$B = 1.7$	0.174	1.889	10.9
S3	Strengthened	Horizontal and X-brace FRP	$B = 1.7$	0.174	2.136	12.3

contribution of torsional DOF in the drifts of the structure is too small in comparison with translational movements.

As for the FRP-strengthened structures, the peak medians obtained were equal to 0.71, 0.68, and 0.57% for the H-FRP, H&V-FRP, and H&X-FRP, respectively. It was found that the FRP retrofitting performed well in controlling the maximum inter-story drift ratio during ground motions at the design level. A significant decrease in the peak median was observed in comparison with the original structure with $B = 1.7$, starting from a 14% decrease in the H-FRP structure and peaking at a 31% decrease in the H&X-FRP structure. This can be justified by the effect of FRP composites on increasing the shear strength of original walls and coupling beams through wrapping the members. Increasing the shear strength can alter the failure mode of each part to a more ductile mode with higher energy dissipation. Subsequently, it limits the interstory drift ratio of the RC core wall, which leads to more ductile behavior. Nevertheless, the investigated retrofitted schemes can postpone the stiffness deteriorations, which points out better damage and drift control on retrofitting. Moreover, the interquartile range (Q3-Q1) was found to be smaller in the FRP-strengthened

structure in comparison with the original structures. For instance, this can be clearly observed in Fig. 16(f) for H&X-FRP. Results of initial pushover analyses showed FRP retrofitting using FRP X-braces to be the most efficient method, capable of enhancing both the strength and the ductility of RC walls. Similar results were observed by El-Sokkary et al. (2012) and El-Sokkary and Galal (2013).

Story Shear Demand Envelope

The effect of torsional sensitivity on story shear demand was investigated in the current study. As shown in Fig. 17(a), the story shear force profiles over building height were shifted rightward by increasing the torsional sensitivity of the structure. The story shear was normalized by the peak base shear in the structure with $B \approx 1.0$. The maximum base shears were increased by 34 and 63% for the structures with $B = 1.7$ and $B = 2.5$, respectively.

As for the effectiveness of FRP strengthening, the strengthened structures were also investigated and the results are shown in Fig. 17(b). By normalizing the results by the peak base shear in

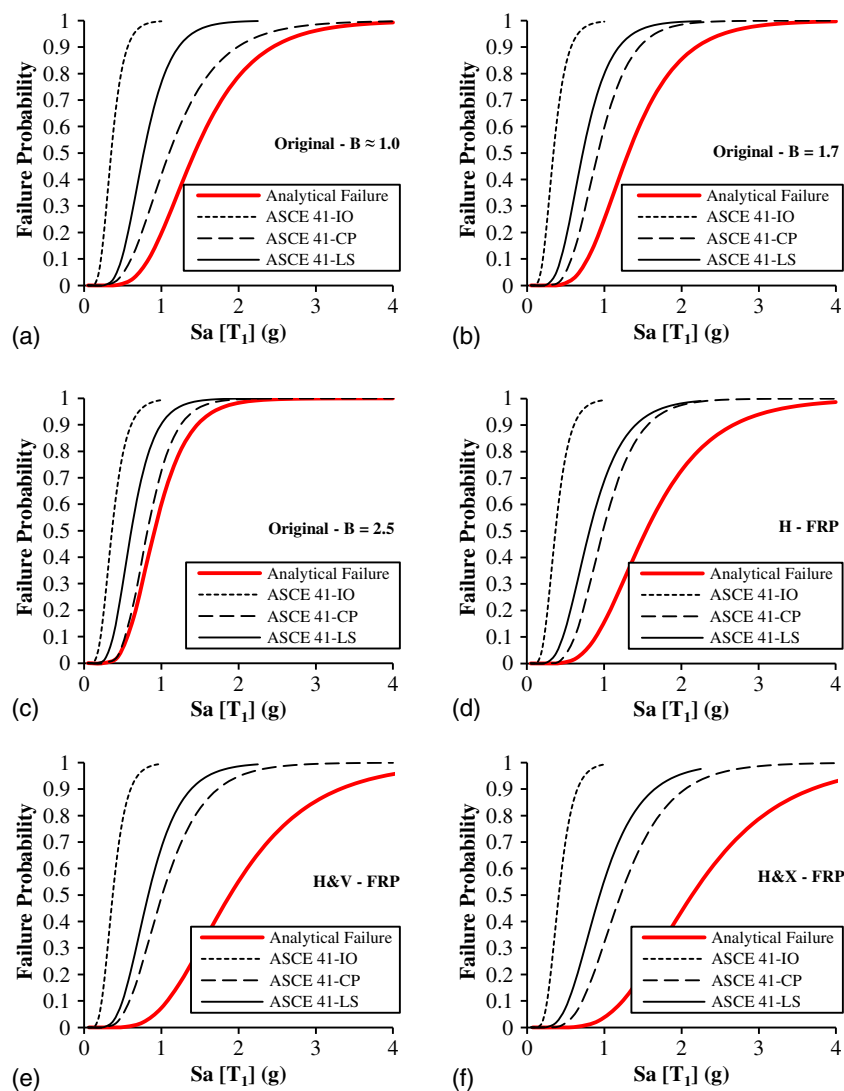


Fig. 15. Fragility curves for different performance levels of original and strengthened buildings

the case of $B = 1.7$, it was found that FRP retrofitting with horizontal wrapping increased the base shear by 7% whereas the H&V-FRP retrofitting resulted in an increase of 8%. The peak base shear in the H&X-FRP structure increased by 23%.

Regarding the increase in the base shear demand of the structure, FRP retrofitting enhancing both the strength and the ductility of the RC core wall system without a noticeable increase in the initial stiffness of the structure. Increasing the strength, however, causes an offset in stiffness degradation of the wall system and leads to a stiffer response in the hardening region (between the

yield point and the peak strength). Hence, although retrofitting increases the capacity of the system, it also increases the shear demand.

Failure Modes

Different failure modes, previously mentioned, were considered during the dynamic analyses and in postprocessing to find the most common failures and the most critical regions in the investigated buildings. The failure mechanisms were considered in four groups: shear/flexural failure, confined concrete crushing, unconfined concrete crushing, and steel rebar buckling/fracturing. Axial failure was excluded because it was found to be dominant in none of the models in the current work.

Fig. 18 shows the results of failure distribution over the building height. As can be seen, shear failure of the wall usually happened in the first story of the building and concrete crushing failures were distributed between the base and the third floor. The story level of the steel rebar buckling/fracture failure mode ranged from the first floor up to the seventh floor. The formation of failure along the height of the building demonstrates the contribution of higher modes to building seismic response under selected ground motion records. It confirms the findings of Boivin and Paultre (2010) and

Table 8. Median Structural Capacity Associated with Drift Limit Proposed by ASCE41-13

Archetype identifier	Description	Limit state		
		IO	LS	CP
O1	Original— $B \approx 1.0$	0.35	0.77	1.1
O2	Original— $B = 1.7$	0.34	0.72	0.92
O3	Original— $B = 2.5$	0.36	0.61	0.82
S1	H—FRP	0.37	0.81	0.99
S2	H & V—FRP	0.37	0.83	1.01
S3	H & X—FRP	0.41	0.94	1.21

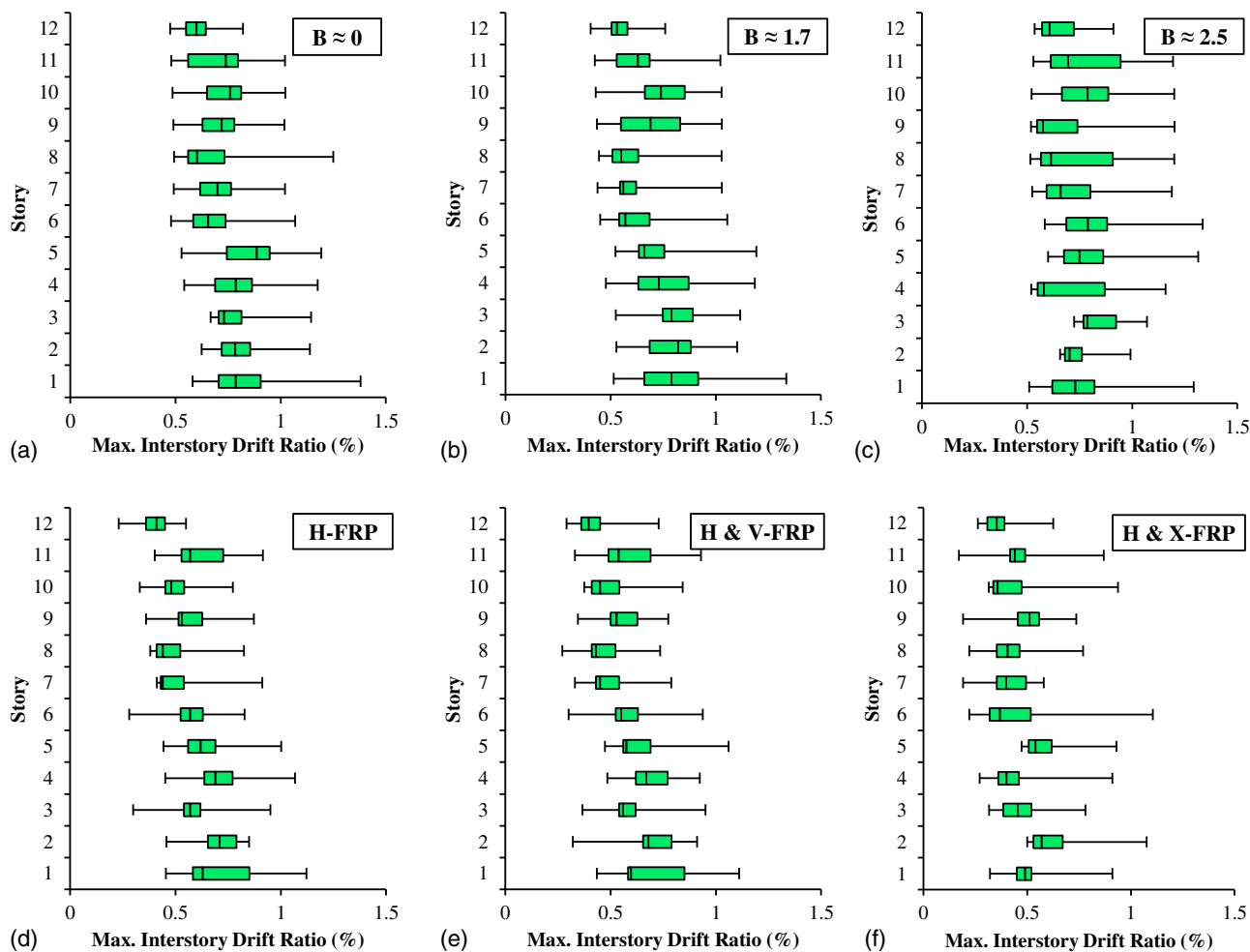


Fig. 16. Interstory drifts of 12-story buildings for all 50 records at design levels

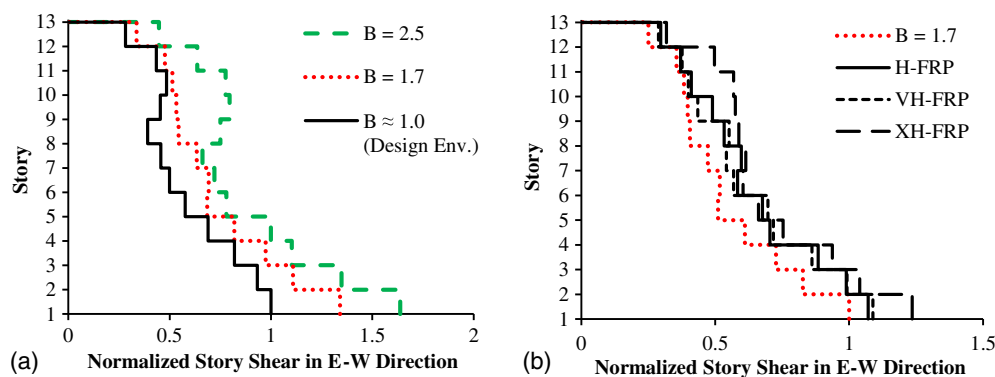


Fig. 17. Story shear force profiles over building height in the east-west direction

highlights the influence of the dynamic shear amplification factor recently introduced in CSA A23.3-14 to account for inelastic effects due to higher modes.

Such effects from the higher modes of multistory structures can lead to increases the shear forces beyond the design level in the different stories. Hence, shear forces and bending moments in the upper floors may exceed the calculated design levels, and inelastic rotation can consequently occur in a region without properly implemented ductile detailing. Therefore, some failure modes

might not necessarily occur at the base of the structure. This was also observed by Tremblay et al. (2008).

The variations in the story level of mechanisms for different failure modes are summarized in Table 9. According to the tabulated data, the distribution of different failure modes over the building height was quite similar in the buildings with different torsional sensitivity. As for the strengthened building, however, FRP retrofitting led to shifting of the concrete crushing failures downward to the first story. Moreover, the shear/flexural failures were slightly

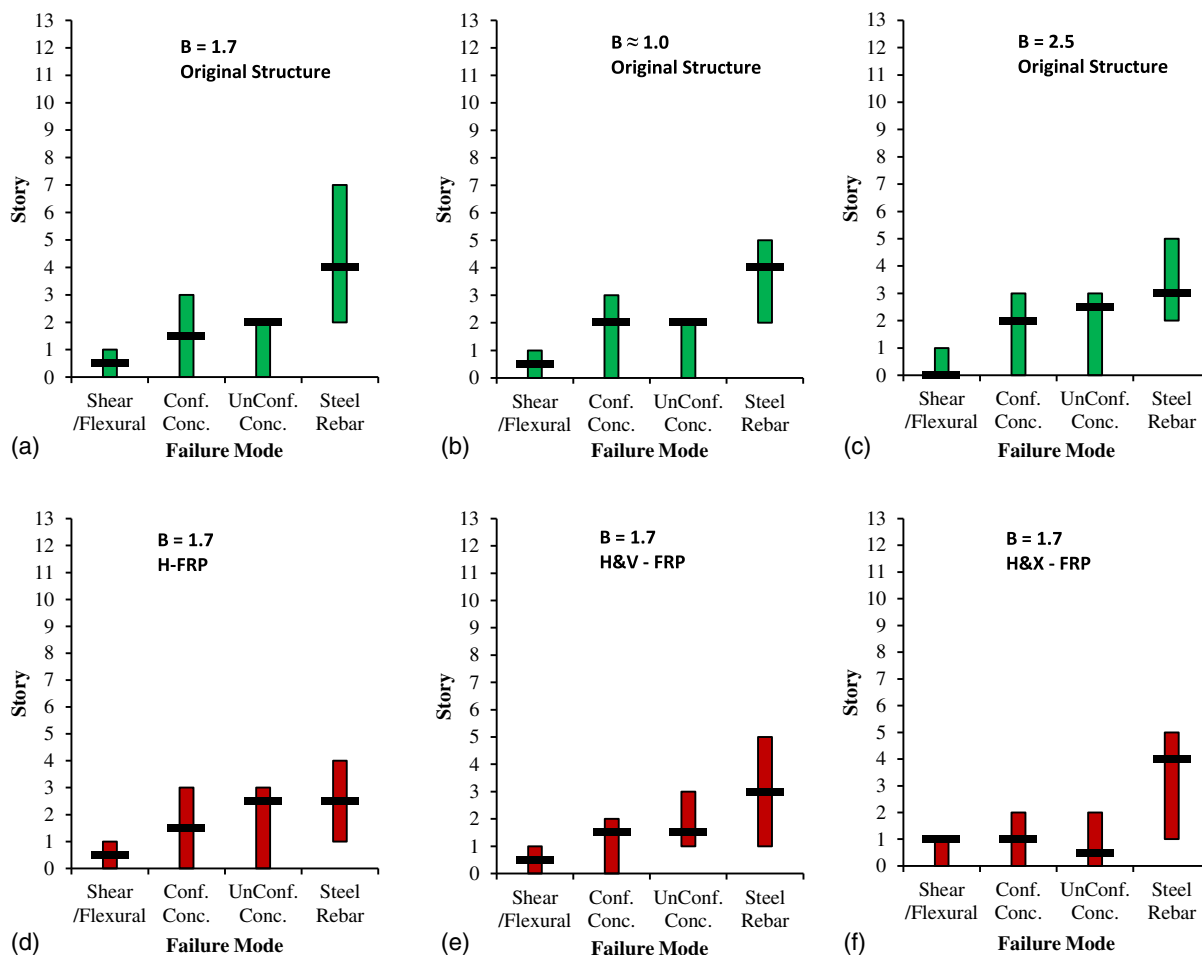


Fig. 18. Failure distribution over building height

shifted upward to the top of the first story after retrofitting with H&X-FRP.

Fig. 19 shows the distribution of each predicted collapse mechanism of the 12-story coupled core wall system in all models. As is clearly depicted, the most common failure mode was shear/flexural failure (55% median), and the least common was steel rebar failure (8% median). Table 10 presents the tabulated data for different building configurations, in which similar trends for all of the buildings can be seen. The proportion of shear/flexural failure was increased in buildings with higher torsional sensitivities, ranging from 56 to 68% in the buildings with $B \approx 1.0$ and $B = 2.5$, respectively. A slight decrease in the proportion of shear/flexural failure was also observed, from 54% in the buildings with H-FRP and H&V-FRP to 50% in the building with H&X-FRP.

Table 9. Variations in Story Level of Mechanisms for Different Failure Modes

Archetype identifier	Description	Median story level of failure			
		Shear/flexural	Confined concrete	Unconfined concrete	Steel rebar
O1	$B \approx 1.0$	0.5	2	2	4
O2	$B = 1.7$	0.5	1.5	2	4
O3	$B = 2.5$	0	2	2.5	3
S1	H-FRP	0.5	1.5	2.5	2.5
S2	H&V-FRP	0.5	1.5	1.5	3
S3	H&X-FRP	1	1	0.5	4

Certain limitations must be recognized in evaluating the results of this study which offer suggestions for future studies. First, IDA analyses were conducted only in the east-west direction (coupled direction of the wall) and the effects of bidirectional seismic excitations were not considered. Second, the modeling approach used here did not accurately capture the postpeak strength degradation of the wall specimens. Third, perfect anchoring for the FRP strips was assumed and failure of the FRP connections was not considered. The results of the study were limited to a building

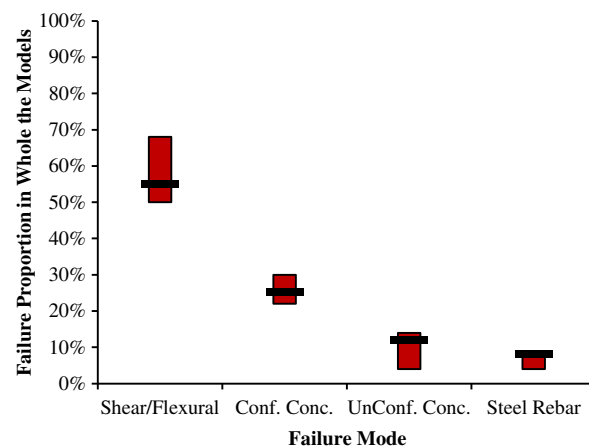


Fig. 19. Distribution of different failure modes among the models

Table 10. Predicted Collapse Mechanism of the 12-Story Coupled Core Wall System

Archetype identifier	Description	Shear (%)	Failure type distribution		
			Confined concrete (%)	Unconfined concrete (%)	Steel rebar (%)
O1	$B \approx 1.0$	56	22	14	8
O2	$B = 1.7$	58	22	12	8
O3	$B = 2.5$	68	24	4	4
S1	H-FRP	54	30	12	4
S2	H&V-FRP	54	26	12	8
S3	H&X-FRP	50	28	14	8s

designed and analyzed for Montreal, Canada, only. This research needs to be extended to other seismic regions in order to generalize the findings.

Conclusions

This study provides results from IDA analysis of coupled RC core walls using a nonlinear macromodeling approach to investigate the collapse capacity of coupled RC cores. The modeling approach was initially developed by other researchers. However, modifications were proposed in the current study to consider the effectiveness of FRP strengthening on the collapse response of RC core systems. Using a set of 50 artificial ground motion records, more than 2,400 time history analyses were conducted on nonlinear 3D models of 12-story buildings with different torsional irregularities and strengthening schemes. Results showed the following:

- Based on the good agreement between the results of FE analysis and the experimentally tested C-shaped RC shear walls reported in the literature, the elements and modeling considerations adopted in this study proved to be capable of simulating the behavior of this type of shear wall under cyclic loading; the maximum error percentages for the prediction of lateral load capacity and lateral stiffness using the approach in this paper were 6 and 10%, respectively; moreover, the numerical model performed well in predicting the unloading stiffness of a simulated wall with a maximum error of 7%;
- The case assessed in the current study showed a CMR of more than 5 for the original structure, which passes the acceptable collapse margin ratio (ACMR) proposed by FEMA P695 (FEMA 2009); however, by using a proper strengthening scheme with FRP material, an improvement of up to CMR = 12.3 with flatter fragility functions can be achieved for the structural system; results are in agreement with the work recently published by Gogus and Wallace (2015);
- Torsional sensitivity has a significant effect on the collapse capacity of a building, resulting in up to 8 and 36% decreases by reaching the torsional sensitivities of $B = 1.7$ and $B = 2.5$, respectively; steeper fragility functions were also observed by increasing torsional sensitivity;
- FRP retrofitting was found to be an efficient method for improving the collapse capacity of coupled C-shaped RC cores; three strengthening schemes were evaluated, and X-bracing of walls with FRP strips was found to be the most efficient strengthening layout; these results were previously confirmed by experimental tests on planar RC shear walls by El-Sokkary and Galal (2013);
- The torsional sensitivity of the structure has no significant influence on interstory drift components of the system; FRP retrofitting, however, performed well in reducing interstory drifts (14–31%) and in enhancing system performance; moreover, the fluctuation in drift ratios was lower than the corresponding values in the original structures; and

- Shear/flexural failure was the most common failure mode, and steel rebar failure was the least common; the proportion of shear/flexural failure increased up to 21% in buildings with higher torsional sensitivities.

Appendix. Bond-Slip Model for the Concrete-FRP Interface

Three bond-slip models were proposed by Lu et al. (2005) to consider the concrete-FRP interface. Because of its simplicity, the bilinear bond-slip model was adopted in the current work to create a debonding rule for the interaction between the concrete and FRP strips. The model proposes a set of equations for calculating the relation between the local bond stress τ and the local slip s , described by the following equations:

$$\tau = \tau_{\max} \frac{s}{s_0}; \quad \text{if } s \leq s_0 \quad (8a)$$

$$\tau = \tau_{\max} \frac{s_f - s}{s_f - s_0}; \quad \text{if } s_0 < s \leq s_f \quad (8b)$$

$$\tau = 0; \quad \text{if } s > s_f \quad (8c)$$

where $\tau_{\max}$ = maximal local bond stress; s_0 = local slip at $\tau_{\max}$; and s_f = local slip when bond stress τ reduces to zero. These parameters can be calculated by the following equations:

$$\tau_{\max} = \alpha_1 \beta_w f_t \quad (9)$$

$$s_0 = 0.0195 \beta_w f_t \quad (10)$$

$$s_f = 2G_f / \tau_{\max} \quad (11)$$

where

$$G_f = 0.308 \beta_w^2 \sqrt{f'_c} \quad (12)$$

$$\beta_w = \sqrt{\frac{2.25 - b_f/b_c}{1.25 + b_f/b_c}} \quad (13)$$

where $\alpha_1 = 1.5$ = coefficient in proposed bond-slip models determined by the iterative procedure; β_w = width ratio factor; b_f = FRP strip width; and b_c = concrete segment width, taken equal to the tributary area of each FRP strip.

Acknowledgments

The authors would like to acknowledge the financial support of le Fonds de Recherche du Québec–Nature et Technologies (FRQNT) in a team grant with École Polytechnique de Montréal.

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