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Seismic behaviour of moderately ductile shear walls located in Eastern North America

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Abstract: This paper presents the numerical modelling of scale shake table tests of slender 8-story reinforced concrete (RC) shear wall specimens. The proposed numerical modelling procedures are used to investigate the seismic behaviours of full scale RC shear walls. Firstly, using the shake table feedback signals as input motions, nonlinear time history analyses are carried out using reinforced concrete fibre elements (computer program OpenSees, OS) and the finite element (FE) method (computer program VecTor2, VT2). Good agreements are obtained between the numerical and experimental results. Using the proposed numerical modelling strategy, it is possible to investigate the nonlinear dynamic responses of slender RC wall structures with confidence. Secondly, nonlinear time history analyses of typical moderately ductile shear wall buildings with number of storey ranging from 5 to 25 and fundamental period varying from 0.5s to 3.5s under Eastern North America (ENA) earthquakes using proposed OS and VT2 modelling procedures were conducted. The results confirm deficiencies of NBCC 2010 and CSA 23.3-04 requirements for shear wall design. There are possibilities of shear failure at base and unintended second hinge formation in the upper part of the walls.

Key words: shear walls; finite element; experimental tests; inelastic response; high frequency content earthquakes, moderately ductile, seismic force demand

1 Introduction

RC shear walls are frequently used as seismic force resisting systems in high rise buildings. However, due to the complex behaviours of RC members under seismic loading, many problems have not been completely understood yet. Among them, there are higher mode effects that have been clear acceptance within the engineering community as playing an important role in the seismic responses of structural walls (Tremblay et al. 2001; Panneton et al. 2006; Ghorbanirenani et al. 2009; Boivin and Paultre 2012; Wiebe et al. 2012). Therefore, it is very important to understand the behaviour of RC shear walls considering higher mode effects and evaluate appropriately their responses. Consequently, a shear wall project is being conducted at Ecole Polytechnique de Montreal. In the first stage of the project, Ghorbanirenani et al. (2012) performed shake table tests on two reduced scale specimens, 9 m high, of slender 8-storey moderately ductile RC shear wall. The walls were design in accordance with National Building Code of Canada (NBCC) 2005 (NRCC 2005) and CSA 23.3-04 and subjected to the high predominant frequency ground motions (approximately 10 Hz), typical in ENA. The test results showed significant effects of higher modes on the seismic responses of the tested walls. In particular, the tests confirmed that the shear and flexural demands from the code were underestimated; and inelastic behaviour was observed at the base as well as in the sixth storey of the specimens.

In a second step of the shear wall project, this paper presents the development of constitutive shear wall models that are validated by the tested wall data. Both the finite element method using the computer program Vector 2 (VT2) (Wong and Vecchio 2002) and the fibre element computer program OpenSees (OS) (Mazzoni et al. 2006) are used. VT2 is based on 2D plane stress finite element theory and includes most of the phenomenological features present in RC members. OS is a multi-fibre beam element program based on the Euler-Bernoulli theory. It represents an attractive alternative to (FE) modelling, as it can reproduce the dominant inelastic flexural response anticipated in shear walls. The proposed OS and VT2 modelling procedures are used as representative constitutive shear wall models to investigate higher mode effects on the seismic demand of typical full scale RC moderately ductile shear walls under high frequency content ENA earthquakes.

2 Large scale shaking table test

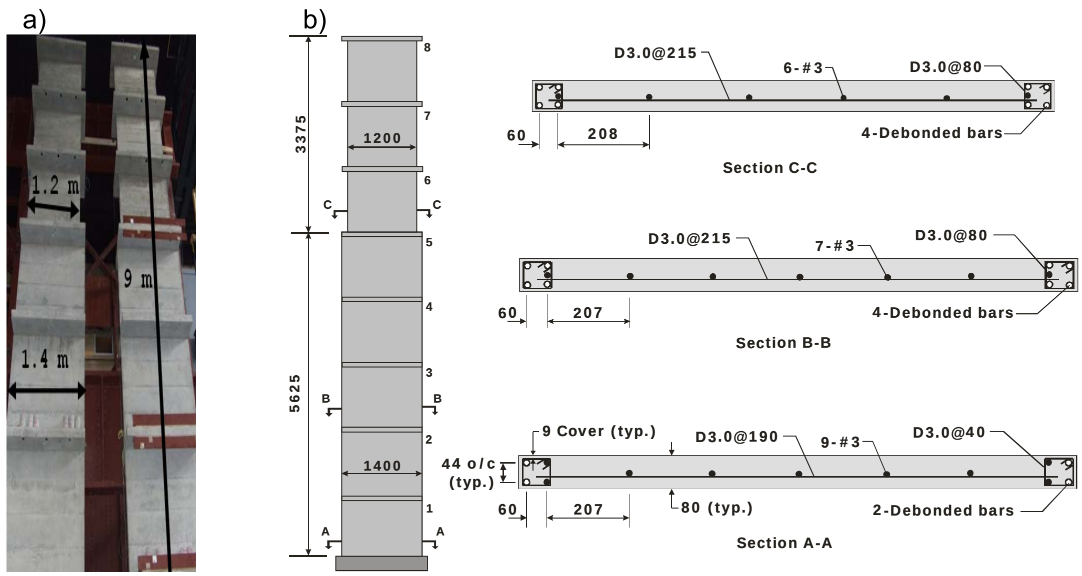

Figure 1: (a) model wall; and (c) cross-section of the model wall.

Two RC walls (W1, W2) were fabricated using a prototype 8-storey building located in Montréal, QC and scaled by a length factor l_r = 0.429. The total height of the prototype building was 20.95 m and the model walls were 9.0 m high. The walls were designed according to NBCC 05 and CSA-A23.3-04 for the moderate ductility category (ductility-related force modification factor R_d= 2.0 with overstrengh-related force modification factor R_o=1.4) assuming a firm site soil class C. The uniaxial seismic simulator of Ecole Polytechnique has a payload capacity of 15 tons and 3.4 m x 3.4 m plan dimension, 60 kN floor seismic weights were installed on multi-level hinged posts beside the shake table, at every level of the test specimen. The inertia loads were transferred by rigid links that connected the wall to the seismic weights. Details of the test setup and experimental program are presented in Ghorbanirenani et al. (2012) as indicated in Figures 1a and b. Figure 1b indicates that 639 mm^2 to 426 mm^2 of longitudinal reinforcement were used along the height of the walls.

In the test program, the two walls were subjected to stepwise incremented ground motion excitations. An ENA earthquake Mw7.0 at 70 km simulated ground motion time history was selected and spectrally matched to the NBCC 2005 uniform hazard design spectrum (UHS) for Montreal. The ground motions were scaled by a factor of a_r=2.65 and the time contracted by a factor of t_r=0.403 for similitude requirements with the prototype.

Wall W1 was tested under 40% (elastic), 100% and 120% of the NBCC ground motion design intensity. Wall W2 was tested under 100%, 120%, 150% and 200% of the design ground motion intensity. In this paper, only the results of the initially undamaged Wall W2 when subjected to NBCC design ground motions are considered. Complete tests results could be found in Ghorbanirenani et al. (2012).

3 Constitutive RC shear wall models

3.1 Fibre element model

Nonlinear beam-column elements with fibre discretisation of the cross-section are used in the OS program to model the tested walls. Ten and sixty concrete fibres are used along the thickness and length, respectively The integration along the element is based on the Gauss-Lobatto quadrature rule. In this study, 5 integration points are selected along the height of the element. The measured material properties of the wall presented in Ghorbanirenani et al. (2012) are used in the numerical model. The Concrete 02 model (Mazzoni et al. 2006) is used for concrete fibres. The confinement effects on the concrete response in the regions of concentrated reinforcement were accounted for by increasing both the compressive strength and strain by 20%. For steel, the steel02 model (Mazzoni et al. 2006) that follows the Giuffré-Menegotto-Pinto model without considering the isotropic strain hardening is used. The shear deformation is modelled by including linear shear cross-sectional stiffness coefficients. The axial force (90.7 kN) induced by post-tensioned vertical bars is considered as a static force and modelled as a point load at the top of the wall.The self-weight of the wall is considered when determining both the static axial and seismic loads. The seismic mass is lumped at each floor. P-Delta effects are not considered in the analysis as they have been shown to be insignificant in previous studies for wall design in accordance with Canadian code (Tremblay et al. 2001). In numerical simulations, the applied seismic excitations correspond to the measured shake table acceleration signals during the tests. Therefore, the shake table is not included in the numerical models. The integration of the equations of motions uses the Newmark method with $\beta = ¼$ and $\gamma = ½$ with a time step of 0.005 s.

A complementary study (Luu et al. 2013a), that is not discussed in this paper, showed that using OS model with (i) 2% Initial stiffness proportional Rayleigh damping for first two modes; (ii) 5%-30% of gross shear stiffness at base and in the middle height; and (iii) no tension stiffening effect (TSE) give a good agreement between numerical and experimental results. Summery of OS model parameters used are shown in Table 1.

Table 1: Selected parameters for the OS model.

	OS Model parameters
Concrete	Concrete 02
	Confined strength: increase 20%
Steel	Steel 02
Damping (%)	$[C]_{ini}$[1]; 2% for modes 1 and 2
Shear stiffness	5%-50% at base and in the middle height
TSE [2]	Not included

[1] $[C]_{ini}$ = Initial stiffness proportional Rayleigh damping; [2] TSE = Tension stiffening effects

3.2 Finite element model

VT2 is based on the Modified Compression Field Theory and the Disturbed Stress Field Model for the nonlinear finite element analysis of reinforced concrete membrane structures (Vecchio and Collins 1986; Vecchio 2000). VT2 is used to develop the finite element model of the tested walls. Two-dimensional plane stress rectangular elements with smeared steel reinforcement are used, except in the flange areas with lumped reinforcement . The material properties used in the FE models corresponded to the as-built material properties measured in the laboratory at the time of the tests, as described by Ghorbanirenani et al. (2012). The pre-peak compression response of the concrete is based on the Hogdnestad parabolic model (Wong and Vecchio 2002), whereas the post-peak response followed the modified Kent-Park model (Wong and Vecchio 2002). The hysteretic response of the concrete is set according to Palermo and Vecchio (2002) (with decay). The tensile softening branch descends linearly from the cracking stress and strain to zero stress at the characteristic strain (Wong and Vecchio 2002). The slip distortion is taken into account according to the stress-based model by Vecchio and Lai (Wong and Vecchio 2002). The concrete strength enhancement due to confinement is considered using the Kupfer/Richart model (Wong and Vecchio 2002) for concrete located in the region of concentrated reinforcement. The hysteretic model of the reinforcement is made according to the Seckin model (with the Bauschinger effect) (Wong and Vecchio 2002). The seismic mass of each floor is distributed at the nodes of that floor. The mesh is made of 1113 rectangular elements with the horizontal size of 100 mm, a finer mesh using 50 mm horizontally does not significantly improve the results. Similar to OS model, the axial force (90.7 kN) induced by post-tensioned vertical bars is considered as a static force and modelled as a point load at the top of the wall, and the self-weight of the wall is considered when determining both the static axial and seismic loads, the P-Delta effects are neglected, and the Newmark method is used for the integration of equation of motion.

A complementary study (Luu et al. 2013a) showed that using VT2 model with (i) 1.5% initial stiffness proportional Rayleigh damping for modes 1 and 3 and (ii) no TSE give a good agreement between numerical and experiment results. Summary of the VT2 model parameters used are shown in Table 2.

Table 2: Selected parameters for the VT2 model.

	VT2 Model parameters
Concrete	compression pre-peak: Hogdnestad parabolic
	compression post-peak: modified Kent-Park
	Confined strength: Kupfer/Richart
	Hysteretic response: Palermo 2002 (with decay)
Steel	Hysteretic response: Seckin (with Bauschinger effect)
Damping (%)	$[C]_{ini}$ [1] ; 1.5% modes 1 and 3
TSE [2]	Not included

[1] $[C]_{ini}$= Initial stiffness proportional Rayleigh damping; [2] TSE = Tension stiffening effects

3.3 Numerical versus experimental results

In this section, the study is restricted to W2 under different EQ intensities.Table 3 presents the dynamic characteristics and peak responses obtained from both OS and VT2 models in comparision to experiment for W2. The first natural periods are generally close to the values obtained from the free vibration response of the wall specimens in free vibration impact tests. The shorter values obtained from the OS model are due to the use of an effective shear stiffness. As nonlinearity increased in the VT2 and OS models, the natural periods of the walls elongated approximately in the same proportion as the experimental values. This indicates that the cracking and level of damage in the wall are well captured by the constitutive RC model.

Although a 1.5% constant viscous damping ratio ξ for the first and third modes was used in the VT2 analyses, the damping calculated using the VT2 model from the free vibration response at the end of each test is significantly larger. That additional damping in the VT2 model is due to the accumulated damage in the steel and concrete materials, the energy dissipation caused by the opening and closing of the cracks, and the tangential crack motions modelled in VT2.

Table 3 presents the maximum top displacements ($\Delta_{max\ (top)}$) for W2 under different EQ intensities. All of the predictions from OS and VT2 are close to the experimental values, especially for the case for W2 under 100% EQ, which was initially undamaged. Figure 2a compares the top displacement histories of W2 under 100% EQ with those obtained from the OS and VT2 models. VT2 predictions in Figure 2a follows the same displacement patterns as those obtained in the experiment. The results of the OS model shown in Figure 2a also agree well with the experimental values.

In Table 3, the shear forces obtained from both OS and VT2 models are in good agreement with the experimental results, especially at the base. The moments obtained from both models are very similar to the experimental results. Figures 2b and c compare the shear and moment envelopes along the heights of W2 between the experiments and the numerical models under 100% EQ. For W2, which was initially undamaged, VT2 provides an excellent distribution of the shear forces (Figure 2b). The moment distributions obtained from both OS and VT2 in Figures 3b and c are nearly identical to the test values. VT2 slightly underestimates the value of the base moment, but it predicts an excellent moment distribution pattern in the upper part of the wall (Figure 2c). The OS predictions of the shear force envelope distribution along the wall height in Figures 2b and c for W2 are similar to the experimental values. Due to higher mode effects, the base shears in the walls were amplified in the tests and were much higher than the values predicted by the response spectrum analysis using the feedback input-response spectrum(V_f) (Figure 2b). These effects are captured very well by both the OS and VT2 numerical models.

Figure 3 presents comparisons of the moment-rotation responses obtained from the models and experiments for W2 under 100% EQ. In both walls, inelastic rotations could be observed at the base and at the 6th level. Both the VT2 and OS models yield flexural stiffnesses that are close to the measured values. In Figures 3a and c the OS results match well those of the test with respect to hysteric responses at the 6th level and at base, respectively. The computed force demand and rotational ductility are very close to the test results. VT2 also provided a good match to the experimental hysteretic moment-rotation curves (Figures 3b and d). Table 3 presents the peak rotational ductility demand predicted by both OS

and VT2 model under different EQ intensities for W2. The results show insignificant difference between numerical predictions and experimental results.
The VT2 model has the ability to predict cracking due to shear and bending, the interaction of these cracks in concrete members, and the shear deformation responses, including the reduction in shear stiffness due to bending and shear cracks. This stiffness degradation can be reproduced well with the VT2 model, and this is evident considering predictions of the relations between moments and rotations for W2 at the 6th storey (Figure 3b).

Table 3: Dynamic characteristics and peak responses of W2 from experiment and numerical modellings.

		W2			
EQ Intensity		100%	120%	150%	200%
Parameter T_1(s)[1]	EXP	0.96	1.00	1.03	1.31
	OS	0.81	0.87	1.15	1.24
	VT2	0.94	0.97	0.98	1.03
ξ(%)[1]	OS	4.5	3.5	2.1	1.6
	VT2	4.4	4.6	4.8	5.6
$V_{b\ max}$ (kN)	EXP	140	140	172	183
	OS	141	149	165	187
	VT2	136	135	175	182
$V_{6\ max}$ (kN)	EXP	40	45	46	66
	OS	45	59	85	77
	VT2	43	56	60	57
M_{max} (kN.m)	EXP	250	225	243	253
	OS	243	249	258	260
	VT2	221	213	221	227
$\Delta_{max\ (top)}$ (mm)	EXP	31	38	52	71
	OS	34	39	47	63
	VT2	37	46	51	53
$\mu_{\theta 6}$	EXP	1.3	1.5	3.2	4.8
	OS	1.3	2.0	2.7	4.2
	VT2	1.4	2.8	3.2	3.9
$\mu_{\theta b}$	EXP	1.2	1.1	1.6	1.7
	OS	1.3	1.1	1.8	1.8
	VT2	1.3	1.1	1.2	1.2

(1) Free vibration in the damaged condition

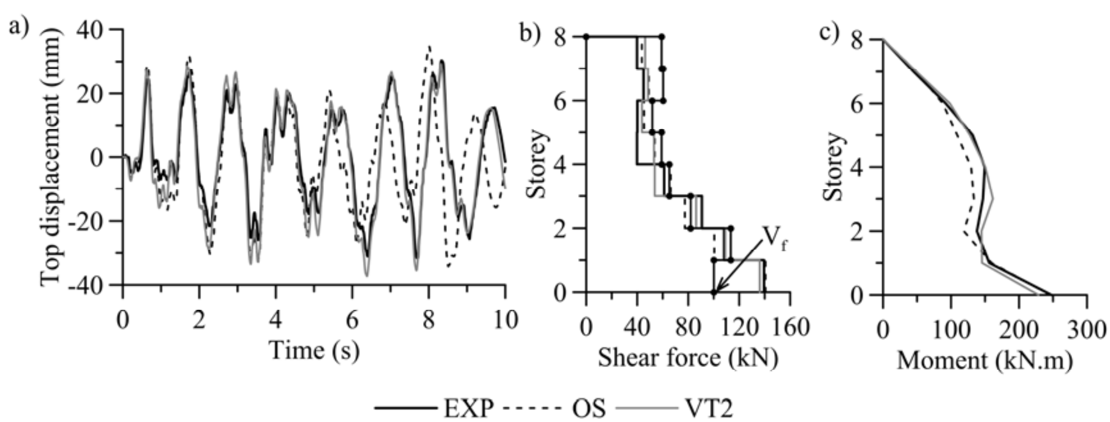

Figure 2: OS model VT2 model vs. experiment under 100% EQ intensity for W2: (a) lateral top displacement time history; (b) moment envelopes; and (c) shear force envelopes.

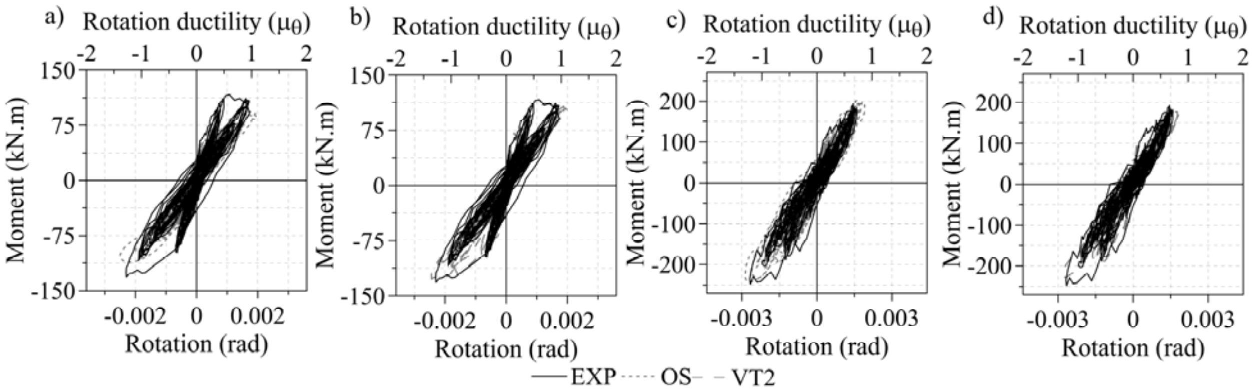

Figure 3: Moment-rotation responses of W2 under 100% EQ: (a) OS vs. the test at the 6th level; (b) VT2 vs. the test at the 6th level; (c) OS vs. the test at the base; and (d) VT2 vs. the test at the base.

4 Assessment of seismic behaviour of ENA moderate ductile shear wall in NBCC 2010 and CSA 23.3-04

Some authors (Panneton et al. 2006; Velev 2007; Ghorbanirenani et al. 2009; Boivin and Paultre 2012) showed that the current Canadian code underestimates structural wall shear forces and bending moment demand. However, these studies were restricted to ductile shear wall (Panneton et al. 2006; Ghorbanirenani et al. 2009; Boivin and Paultre 2012), or moderately ductile shear wall using simple finite element structural analysis program with lumped plasticity beam elements (Velev 2007). Especially, none of these study used numerical models that were validated by large scale shaking table tests. In the following sections the experimentally validated numerical models developed in section 3.1 and 3.2 are used to assess the seismic safety of of RC shear wall design according to NBCC 2010 (NRCC 2010) and CSA 23.3-04 located in ENA.

Table 4: Parameters of considered walls

n	Height (m)	T (s)	Wall cross-section type	Site class	Over-strength factor, γ_w	$P/(A_g f_c')$ (%)	Computer program
5	17.5	0.5	|	D	1.2	5	OS
		1.0	|	D	1.2	12	OS
10	35.0	1.5	|	D	1.2	7	OS
		2.0	|	D	1.2	10	OS
15	52.5	2.0	|	D	1.6	7	OS
		2.5	|	D	1.6	8	OS and VT2
20	70.0	2.5	[	D	2.0	9	VT2
		3.0	[	D	2.0	14	VT2
25	87.5	3.0	[	D	2.0	11	VT2
		3.5	[	D	2.0	15	VT2

4.1 Studied walls

Ten different cantilever walls were designed according to the NBCC 2010 (NRCC 2010) and CSA 23.3-04 requirements. The number of storey (n) varied from 5 to 25 and fundamental period (T) ranged from 0.5 s to 3.5 s. The studied walls are located in the Montreal region on soil class D. The other design input parameters such as wall length, wall cross section, overstrength factor (γ_w), axial load level showing by the ratio of axial loading at base and product of compressive concrete strength and base wall cross section ($P/f''_c A_w$), and wall cross sections were selected considering the feasibility of construction conditions (Table 4). For the shear force, the wall designs were done according to CSA 23.3-04 for moderately ductile shear wall by amplifying calibrated base shear from modal analysis and static force equivalent procedure, V_d, by the over-strength factor, γ_w. Regarding the moment design envelope, there is no requirement for the upper part of the wall for moderately ductile shear wall in CSA 23.3-04, thus the moment envelope in the upper part is taken directly from elastic modal analysis results.Twelve ground motions were selected and scaled according to the recommendations from Atkinson (2009). The mean acceleration response spectra of the scaled ground motions were scaled to get the mean spectra being in good agreement with the design spectrum prescribed in NBCC 2010.

Table 5: Selected shear stiffness and damping model for OS models of considered walls

Storey	Site Class	Shear stiffness (%)	Damping (%)
n=5;T=0.5s, γ_w =1.2	D	19% at base; 5% at 3rd storey	$[C]_{ini}$[1]; 2% for modes 1,2
n=10;T=2.0s, γ_w =1.2	D	50% at base ; 20% at 5th, 6th and 7th storeys	$[C]_{ini}$[1]; 1.5% for modes 1,3
n=15, T=2.5s, γ_w =1.6	D	18% at base; 20% at 7th and 8th storeys	$[C]_{ini}$[1]; 1.5% for modes 1,2

[1] $[C]_{ini}$: Initial stiffness proportional Rayleigh damping

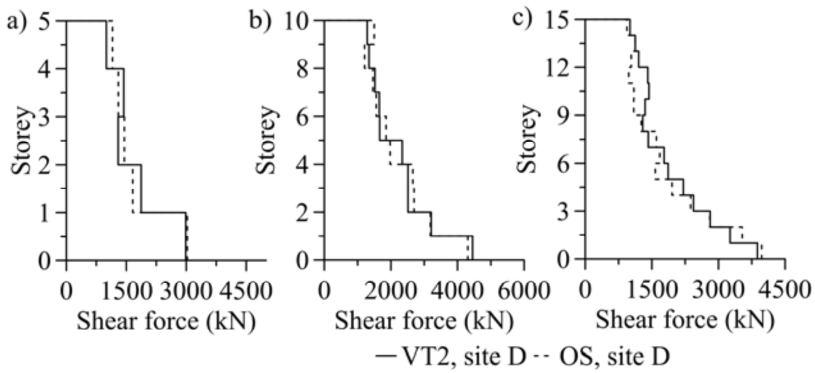

Figure 4: Shear force distribution of OS model predictions calibrated basing VT2 model predictions for site class D: (a) 5 storeys, T = 1.0 s, γ_w = 1.2; (b) 10 storeys, T = 2.0 s, γ_w =1.2; (c) 15 storeys, T= 2.5 s, γ_w = 1.6.

4.2 Constitutive shear wall models

Back analyses of the shake table tests in section 3 showed that both numerical models, OS and VT2, are able to reproduce adequately the experimental data. In particular, the models are capable of adequately predicting higher mode effects resulting in amplification of base shear, second hinge formation in the upper wall region, and redistribution of shear forces.These OS and VT2 wall models were employed in this study. Most of the analyses were conducted with the OS models because they require significantly less computational resources than the VT2 models. The OS models were developed based on the response obtained from VT2 models considering the latter as benchmarks for developing OS models. The details are as follows:

- Develop a VT2 model using the VT2 modeling assumptions validated from the experiments.
- Run VT2 model to obtain the top displacement time history and shear envelop distribution.
- Calibrate the damping model in the corresponding OS model to best match the VT2 top displacement time history.
- In the OS model, calibrate the effective shear stiffness at the wall base and mid-height to get the best match the shear force envelope distribution from VT2.

The calibration was done for a sample wall for each building height under one ground motion record, and then the calibrated effective shear stiffness properties of the OS model were then applied to all walls having that height. For the case n ≤ 15, the calibrated OS models prediction (Table 4) show very good correlation to VT2 model predictions (Figure 4). However, for n ≥ 20, the proposed procedure leads to use a very questionable (unrealistic) damping model by selecting modes 1 and 20 for calibrated OS models. This damping model might under-damp the higher mode behaviours of the wall. The misleading OS calibration models for n=20 and 25 are due to complexities of shear and flexure responses of the high rise shear wall, and the nonlinear shear behaviours might not be properly substituted by an effective shear stiffness from OS models. Consequently, the analysis were done by using VT2 models for the cases n=20 and 25 in this study.

4.3 Nonlinear time history analysis results

The mean results for each ensemble of 12 ground motions, as obtained from the OS models for n≤ 15 and VT2 for n ≥ 20, were used to evaluate the seismic force demand for the walls studied except for cases with special notes. A base shear dynamic amplification factor, referred to as ω_v, is defined as the ratio of the base shear forces from nonlinear dynamic analyses, V_{NL}, to the design base shear, V. In here, V is the design base after considering CSA 23.3-04, and is equal to the multiplication between calibrated base shear V_d and over-strength factor γ_w. The rotational ductility at each storey, μ_θ, was determined as the ratio of the computed rotational demand to the yield rotation at that storey. The yield rotation was evaluated using the method presented by Paulay and Priestly (1992) and used in the previous studies by Ghorbanirenani et al. (2012). OS numerical model parameters developed in section 4.2 were used for pushover analysis to determine yield rotations.

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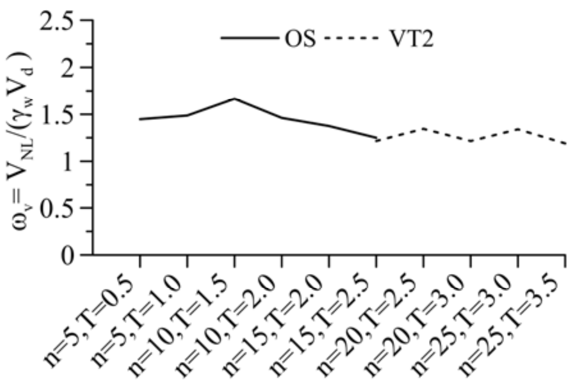

Figure 5: Dynamic base shear amplification factor for studied walls

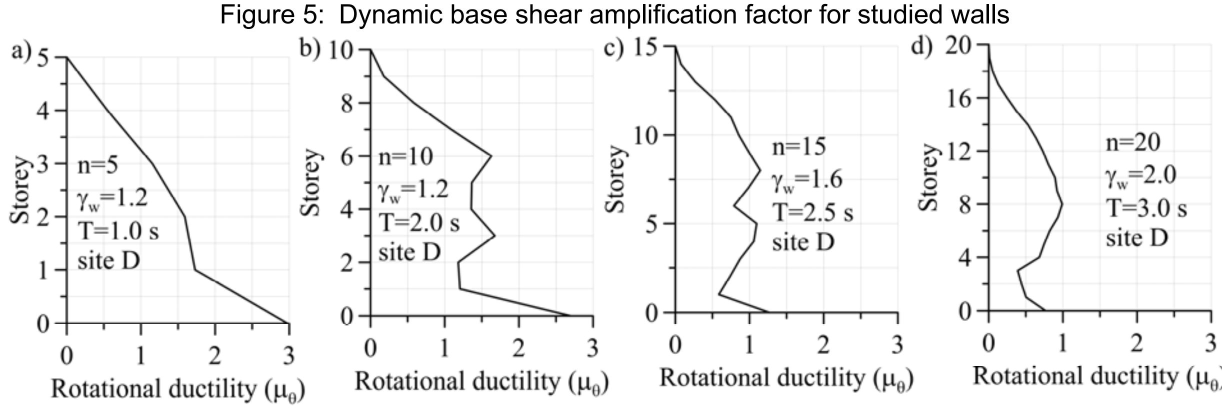

Figure 6: Rotational ductility over wall height: (a) 5-storey, (b) 10-storey, (c) 15-storey; and (d) 20-storey.

Figure 5 illustrates the dynamic base shear amplification factors for studied walls. The results demonstrate the underestimation of current Canadian codes for base shear force predictions. The minimum predicted value is for the case n=25, T=3.5s, but it is still 20% larger than Canadian code prediction. That confirms again the possibility of shear failure at base for the moderately ductile walls design in accordance with Canadian code. Figure 5 also presents a slight decrease of shear amplification factors with the increase of fundamental period T. However, a complementary study shows that this might be due to the effect of overstrength factor γ_w (Luu et al. 2013b).

For the case n=15, T=2.5, Figure 5 shows very good agreement between the predictions of OS and VT2 under 12 selected ground motions. That confirms again the validity of proposed OS model calibration in section 4.2.

Figure 6 presents the rotational ductility demand along the wall for the cases n=5, 10, 15 and 20. The results demonstrate for all cases that the rotational ductility demand could be larger than 1 even in the middle height of the walls. This confirms an unintended plastic hinge formation in the upper part of the wall.

5 Conclusions

In this paper, the results of shake table tests on slender reinforced concrete walls are analysed using numerical simulations. The walls are modelled with the finite element method using the VecTor2 (VT2) and with the fibre element method using the OpenSees (OS) computer programs. Nonliner time history analyses using the proposed these OS and VT2 models for series of full scale RC shear walls are conducted to study the wall seismic behaviours focusing on higher mode effects and high frequency ground motions. The main findings of this study can be summarized as follows:

1) The proposed finite element model (VT2) using initial stiffness proportional to the Rayleigh viscous damping with 1.5% for modes 1 and mode 3 and without considering tension stiffening effects predict well natural periods of the walls, the top displacement time history, moment and shear envelope distributions as well as hysteretic responses for all the test series from undamaged to damaged conditions.

2) The proposed fibre element model (OS) using initial stiffness proportional to the Rayleigh viscous damping 2.0% for modes 1 and mode 2 and without considering tension stiffening effects can predict natural periods of the walls, the top displacement time history, moment and shear envelope distributions and hysteretic responses well for all the test series from undamaged to damaged conditions.

3) The tests showed that a second plastic hinge formed at the 6^{th} level of the walls in addition to the formation of the base hinge due to the higher mode effects. This behaviour is also computed using the OS and VT2 modelling techniques, and there is good agreement in the moment-rotation responses between the test results and numerical model results.

4) Nonlinear time history analysis of typical shear wall buildings with number of storey ranging from 5 to 25 and fundamental period varying from 0.5 to 3.5s under ENA earthquakes using proposed OS and VT2 models confirm the deficiency of NBCC 2010 and CSA 23.3-04 requirements for shear wall design. There is a possibility of shear failure at base and unintended second hinge formation in the upper part of the walls

However, it is noted that the study in this paper is restricted for isolated shear wall under one-dimensional ground motions. The behaviours of RC shear walls located in real buildings with the presences of other structural elements such as beams, columns, slab, and other shear walls subjected to three-dimensional earthquakes could be different and need further investigations.

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