SEISMIC RESPONSE OF MULTI-STOREY REINFORCED CONCRETE WALLS SUBJECTED TO EASTERN NORTH AMERICA HIGH FREQUENCY GROUND MOTIONS

R. Tremblay[1], I. Ghorbanirenani[2], N. Velev[2], P. Léger[1], M. Leclerc[3], S. Koboevic[4], N. Bouaanani[5], K. Galal[6], and D. Palermo[7]

[1] *Professor,* [2] *Graduate Research Assistant,* [3] *Research Associate,* [4] *Assistant-Professor,* [5] *Associate Professor, CGM Dept., Ecole Polytechnique of Montreal, Montreal QC Canada*
[6] *Assistant-Professor, Concordia Univ., Montreal, QC Canada*
[7] *Assistant-Professor, Univ. of Ottawa, Ottawa, ON Canada*
Email: robert.tremblay@polymtl.ca

ABSTRACT :

A research program has been undertaken to better characterizing the contribution of higher modes of vibration on the bending moment and shear force demand on cantilevered reinforced concrete shear walls. Emphasis is put on constructions located in eastern North America. Ground motions in this region have higher dominant frequency, which leads to relatively more significant higher mode response. An example of higher mode contribution is given for two sites in Canada exhibiting different seismic settings. A shake table test program is currently being prepared to examine the influence of damping and shear and flexural stiffness degradation on higher more response. The design of the test specimens and the results from prediction analyses are discussed. A preliminary test program conducted to validate assumptions in the use of reduced-scale wall test models is also presented.

KEYWORDS: Damping, Higher mode response, Flexural ductility, Shear forces, Shear walls

1. INTRODUCTION

Cantilevered concrete shear walls are extensively used in Canada to provide lateral resistance to reinforced concrete as well as steel frames structures. For seismic design, reduced horizontal loads can be used provided that the wall is designed and detailed to exhibit a ductile inelastic flexural response under strong earthquake ground motions. For instance, a plastic hinge region must be created at the wall base where inelastic rotation will take place during severe earthquakes. The design shear forces from analysis are amplified to match the base shear associated with the attainment of the probable yield moment of the wall at its base. This approach has been introduced in the 2005 National Building Code of Canada (NBCC) (NRCC 2005) and the CSA A23.3 (CSA 2004) standard for the design of reinforced concrete structures.

Several past studies have shown that applying these capacity design principles may not be sufficient to guard against shear demand in excess of the design values or plastic rotation above the plastic hinge region, in areas not detailed to sustain inelastic flexural demand (Filiatrault et al. 1994, Tremblay et al. 2001, Priestley and Amaris 2002, Panneton et al. 2006, Sullivan et al. 2006, Boivin et al. 2008). Such undesirable behaviour has been mainly attributed to the contribution of the higher mode of vibration of the wall, even upon yielding of the wall at its base. Dynamic shear magnification factors have been introduced in the New Zealand building code to prevent brittle shear failure (Priestley 2002). No explicit guidance is given in CSA A23.3 to account for this behaviour.

This paper outlines some aspects of a research project that has been undertaken to enhance our understanding of this phenomenon and develop guidelines that would be applicable to Canada and other regions of the world with similar seismicity. Particular interest is devoted to eastern Canada where anticipated seismic ground motions are likely to be richer in high frequency energy, which would lead to a greater influence of the higher modes on wall response. A 15-storey shear wall application is first introduced to illustrate this situation. A preliminary test program that was carried out to validate the use of reduced scale physical models to reproduce the inelastic cyclic flexural and shear responses of R/C wall is then presented and discussed. The design and preliminary analysis of a wall specimen to be used in a shake table test program are also described. The analytical work is also used to highlight other parameters influencing the bending moment and shear demand on shear wall structures, namely the flexural and shear stiffness degradation and damping.

2. HIGHER MODE RESPONSE OF R/C SHEAR WALLS

The influence of the seismicity at the site on higher mode effects is illustrated for a 15-storey reinforced concrete shear wall building located at two different sites in Canada: Vancouver, BC, and Montreal, QC. The hazard at Vancouver, which is also moderate, is representative of that for other cities in the Pacific Northwest region, including Seattle, Portland, and Victoria. The hazard at Montreal is representative of that of many eastern cities located in moderately-active seismic zones, including cities such as Boston, New York. Site Class C corresponding to very dense soil or soft rock was assumed at both sites.

The storey height for the walls is 3 m and the total wall height is 45 m. The walls are designed according to the seismic provisions of NBCC 2005 (NRCC 2005) and CSA A23.3 (CSA 2004). In Vancouver, the wall is designed as a ductile shear wall with an R_d factor of 3.5. The wall in Montreal is a moderately-ductile shear wall with a ductility-related force modification factor, R_d = 2.0. In Montreal, minimum reinforcement requirement often governs and there is no motivation to select a more ductile system. The wall cross-sections and periods of vibration in the first three modes are given in Fig. 1. The periods are those obtained using the cracked cross-sectional properties recommended in CSA A23.3. A dynamic (response spectrum) analysis method is used to determine the bending moment and shear forces along the height of the walls. According to CSA A23.3 provisions, the design moment is maintained equal to the base moment over the height of the plastic hinge zone. Above, the plastic hinge, the moments from analysis are increased to match the design moment at the top of the plastic hinge. For Vancouver, the base shear force is increased to V_p, the shear force associated to the probable moment resistance at the wall base, including strain hardening effects. For Montreal, V_p is associated to the nominal moment resistance. Resistance to V_p is maintained over the plastic hinge length and the shear force demand from analysis above the plastic hinge is amplified accordingly. Although the wall in Montreal is designed with a lower R_d factor, the resulting design seismic loads are lower due to the lower hazard and the wall has a smaller cross-section and longer fundamental period than in Vancouver (Fig. 1).

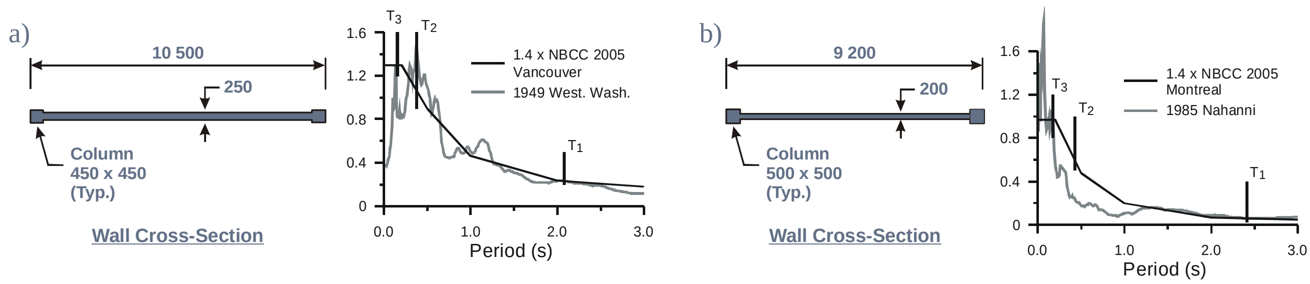

Figure 1. Wall cross-section and acceleration spectra for: a) Vancouver; b) Montreal.

The wall in Vancouver is subjected to a record from the 1949 Western Washington earthquake while an acceleration record from the 1985 Nahanni earthquake is used for the Montreal site. In design, the seismic effects were increased by 40% to account for torsional effects. Only 2D analysis is performed herein and the same amplification factor was applied to both records to achieve a proper supply-demand ratio. The structures were modeled with a cross-section fiber discretization using the wall element in the Ruaumoko computer program (Carr 2004). The Kent-Park model was used for the concrete material and the Al Bermani hysteresis rule was applied for the reinforcing steel. Figure 2 presents the computed time history response at both sites. Only the first 30 seconds of the Western Washington ground motion is presented but the same time scale was preserved for both records in the figure to more easily compare frequency effects.

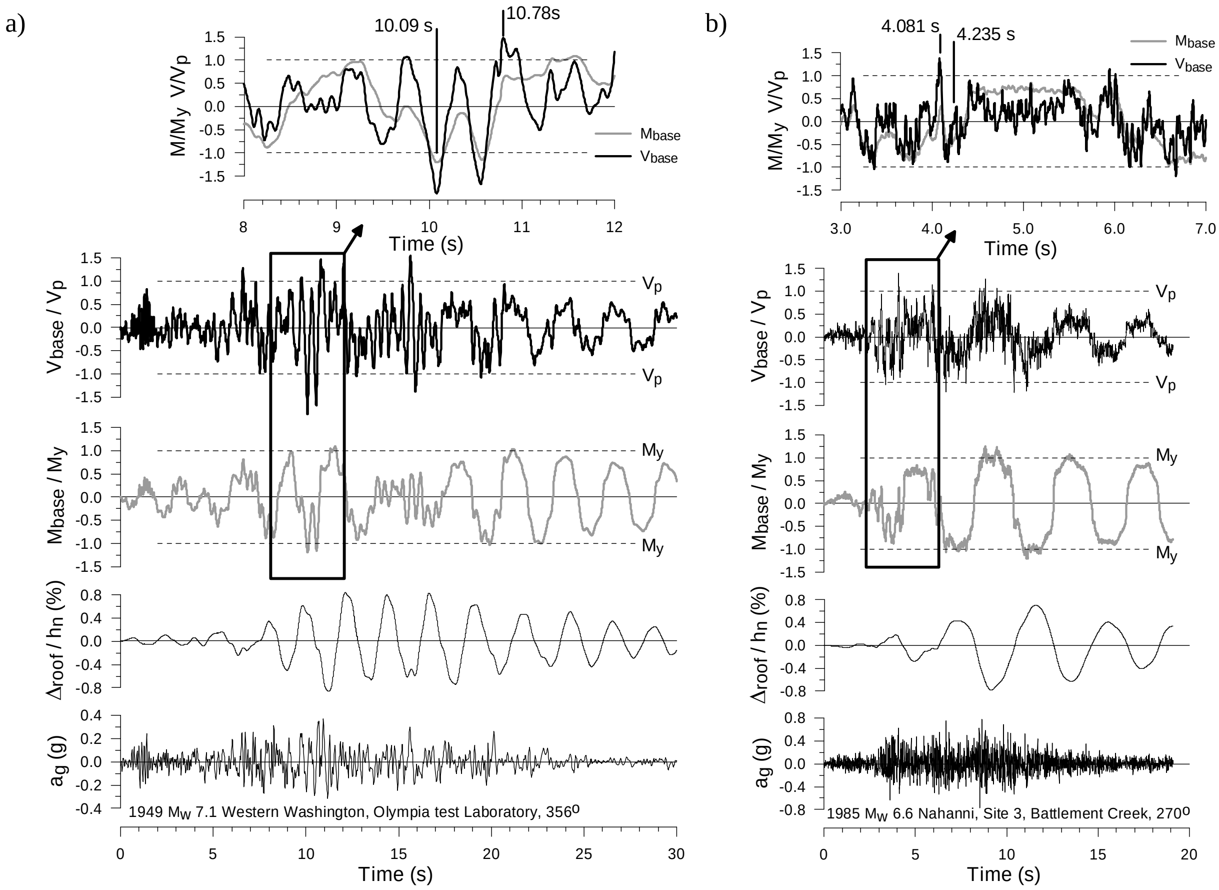

Figure 2 Time history seismic response of the 15-storey walls in: a) Vancouver; b) Montreal.

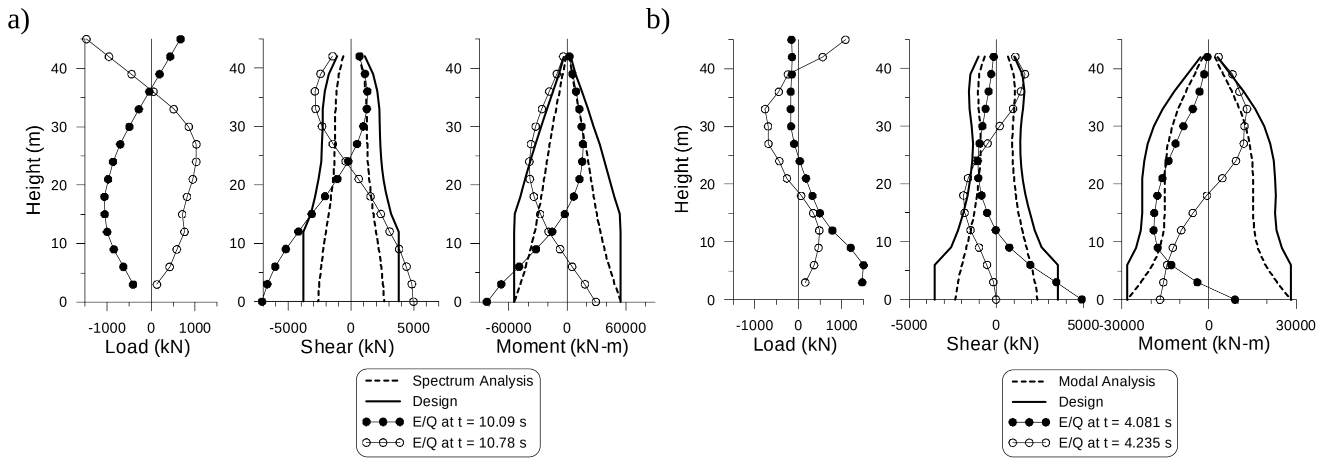

Figure 3 Vertical distributions of the inertial loads, shear forces and bending moments in the 15-storey walls in: a) Vancouver; b) Montreal.

In both cases, roof drift and base moment response are dominated by the fundamental mode. In Fig. 2a, second mode also affects the bending moment and dominates the base shear demand in the first 20 s, and the base shear from the earthquake exceeds several times the design value V_p. In the close-up view of the 8-12 s time interval, the base shear and moment are nearly in phase and V_p is reached first when the moment reaches and exceeds the yield moment, M_y, for the first time, at t = 10.09 s. The distribution of the lateral loads, shear forces, and bending moments along the wall height at that time are presented in Fig. 3a. As shown, the shear force significantly exceeds the design level in the first 10 m at the wall base, which could lead to a brittle shear failure. The demand from the earthquake at t = 10.78 s is also plotted in the figure. The design shear at this time is also exceeded at the base as well as above 30 m in height, and bending moments are greater than the design level in the upper floors. Inelastic rotation could then develop in a region where no ductile detailing would have been implemented. The load patterns at these two particular times clearly indicate that this excessive demand is mainly associated to the second mode wall response. In Fig. 2b, the base shear force in Montreal is associated with high frequency motion corresponding to the second and third modes of vibrations of the wall. This time, the base shear exceeds V_p well before yielding in flexure. This wall possesses flexural overstrength due to minimum reinforcement requirements. Hence, it responded nearly elastically at the beginning of the earthquake, which contributed in attracting higher shear forces. Third mode contribution can be also observed in the inertia load and shear profiles at t = 4.235 s. At that particular time, the base shear is nearly zero but the shear forces and bending moments exceed the design values along the wall height. For this particular wall, third mode response is significant compared to that in the second mode because the ground motion contains relatively low energy near the second mode period, as shown in Fig. 1b.

The magnitude of the higher mode response during an earthquake heavily depends on the wall stiffness. Figure 2 shows that the high frequency motion tends to diminish as inelastic flexural response developed in the wall. In the numerical model used herein, the shear stiffness was assumed to remain constant. In reality, it will also degrade as cracks develop due to flexure and shear. Higher mode contribution is likely to be overestimated with models that only account for inelastic flexural response. However, it is common practice to use Rayleigh damping proportional to mass and initial stiffness with such models, as was the case for the example presented here. Using this damping model tends to artificially attenuate the high frequency response associated to higher modes. These uncertainties in predicting higher mode effects on shear and flexural demand in shear walls were the main motivation for this research project, the main objective being to validate more accurate numerical models that could subsequently be used to better assess this phenomenon.

3. EXPERIMENTAL WORK

3.1. Preliminary Experimental and Numerical Studies

A shake table test program is being prepared to study the response of shear walls under dynamic seismic excitation. The tests will be performed on the uniaxial shake table at the Structural Engineering Laboratory of Ecole Polytechnique of Montreal. This equipment has 15 ton payload capacity and a test height of 10 m. Preliminary studies showed that the test could be performed on a model of a 10-storey prototype building using a scaling factor on length, l_r, of 0.305 to meet the physical constraints of the laboratory (Tremblay et al. 2005). Further investigations indicated that the scaling factor needed to be increased to 0.42 to easily satisfy applicable similitude requirements with available reinforcing steel bar and concrete aggregate sizes. The reference prototype structure was then changed to an 8-storey building.

Prior to initiating the shake table program, testing was carried out to verify the possibility of accurately reproduce with reduced scale models the complex inelastic flexural and shear responses anticipated in actual shear walls. Four specimens were tested: two prototype walls and two 1:2.37 scaled models. The walls were detailed according to the Canadian seismic provisions for ductile walls. Monotonic and cyclic loading protocols were considered for each group. Detail of the study can be found in Gorbanirenani et al. (2008). Figures 4a&b show the test specimens. Excellent agreement was obtained between the prototype and model walls, as exemplified in Fig. 4c for the cyclic tests. This test program also represented an excellent opportunity to validate more refined numerical simulation tools. The Vector2 (VT2) finite element program (Wong and Vecchio 2002) is dedicated to the analysis of concrete structures. It makes use of the Disturbed Stress Field Model (Vecchio 2000) and its scope has recently been extended to cover cyclic and dynamic applications (Palermo and Vecchio 2003). The program could reproduce very well material nonlinearities under monotonic and cyclic loading, including yielding of steel, bar slippage, concrete crushing and cracking, sliding, etc. Strength and stiffness degradation in flexure and shear, as well as the failure mode, could be predicted accurately, as depicted in Fig. 4d. The VT2 program was then used in the design of the shake table specimens.

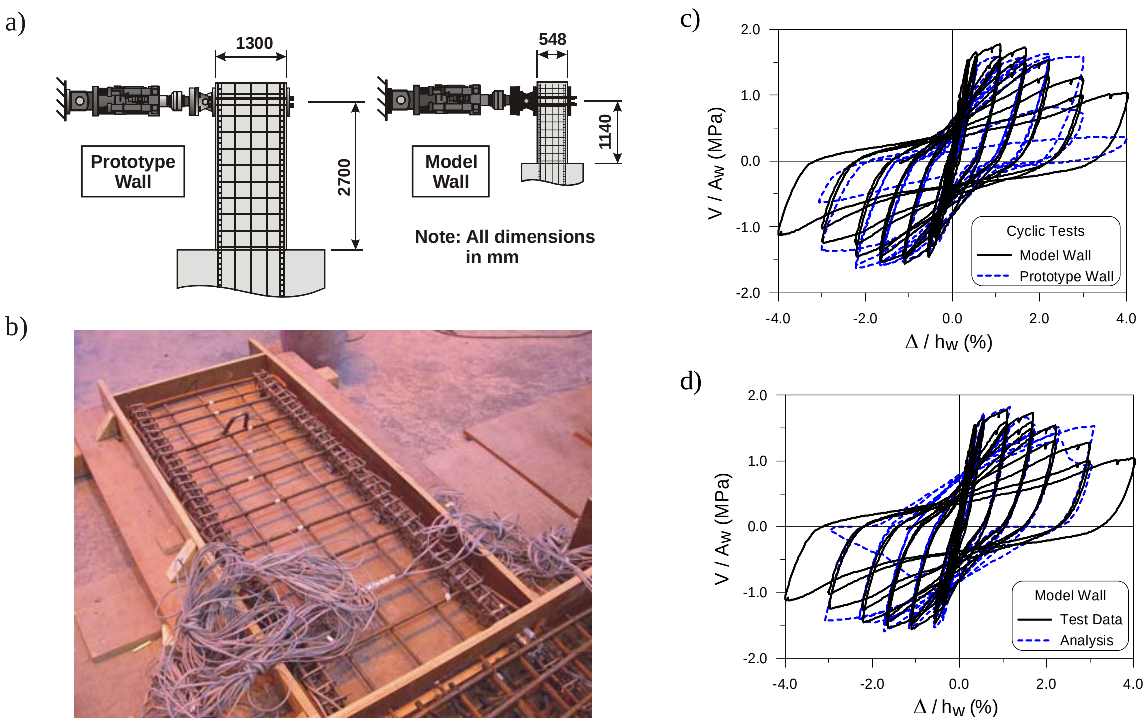

Figure 4 Preliminary testing of shear wall specimens: a) Geometry of the test walls; b) Model wall specimen under construction; c) Measured load-deformation response of the prototype and model walls under cyclic loading; and d) Analytical *vs* measured load-deformation responses of the model wall under cyclic loading.

3.2 Design and Preliminary Analysis of the Shake Table Specimens

The shake table test setup is shown in Fig. 5a. The test specimen is mounted on the earthquake simulator while the seismic weights at each level are supported on an independent structure erected on the strong floor of the laboratory, beside the shake table. The lateral stability is provided by a surrounding steel frame. The reference 8-storey prototype structure has a total height of 20.97 m (8 x 2.621 m) and a scaling factor l_r = 0.429 was adopted to obtain a 9.0 m tall test structure.

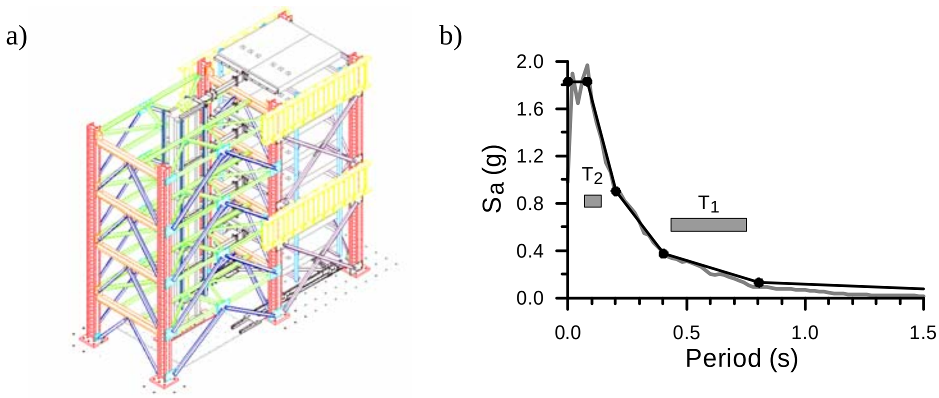

Figure 5 a) Proposed test setup; b) Design and earthquake design spectra.

The artificial mass simulation procedure was used to develop the similitude requirements. The method was modified to also introduce a scaling factor on acceleration, a_r = 2.65, to keep the seismic weight per floor equal to 60 kN. This resulted in scaling factor on time, t_r = 0.403. The test program is conducted for the seismic conditions prevailing in eastern North America and the fundamental period of the prototype structure was selected to fall within the 1.2-2.1 s period range estimated for a typical 8-storey residential building located in Montreal (Panneton et al. 2006). This variation in period resulted from the various assumptions that can be made in the modal analysis. At the model scale, it corresponds to approximately 0.5-0.8 s. Two possible model solutions exhibiting such period values are illustrated in Fig. 6: a simple rectangular wall and an I-shaped wall. Both structures are designed according to the Canadian seismic design provisions for moderately ductile shear walls (R_d = 2.0), assuming a site class C in Montreal. The wall with columns has a higher stiffness. The rectangular wall has a cross-section reduction at the 6th level to encourage exactly meet the required bending moment resistance at that level and examine the possibility of plastic rotation due to higher mode response. An M_w7.0 at 70 km simulated ground motion time history was selected for the test program. It was modified using a loose spectral matching technique to reproduce the design demand for the site. Fig. 5b shows the resulting spectra at the model scale: acceleration and time (period) are scaled. Response time histories obtained with the VT2 program are presented in Fig. 6. In the analysis, axial corresponding to approximately 2.5% of the $A_c f'_c$ has been considered to replicate the future test conditions. As was the case for the sample 15-storey buildings, the roof drift response is governed by first mode whereas second mode dominates the base shear demand. In both cases, the design shear V_p is exceeded and inelastic strain demand (ε > 0.2%) is predicted in the longitudinal reinforcement at the 6th level. Viscous damping was omitted in the VT2 analysis as damping in the test specimens is essentially due to concrete nonlinear behaviour. The decaying free vibration response at the end of the ground motion illustrated in Figs. 7a&b shows that this behaviour is already accounted for in the analysis.

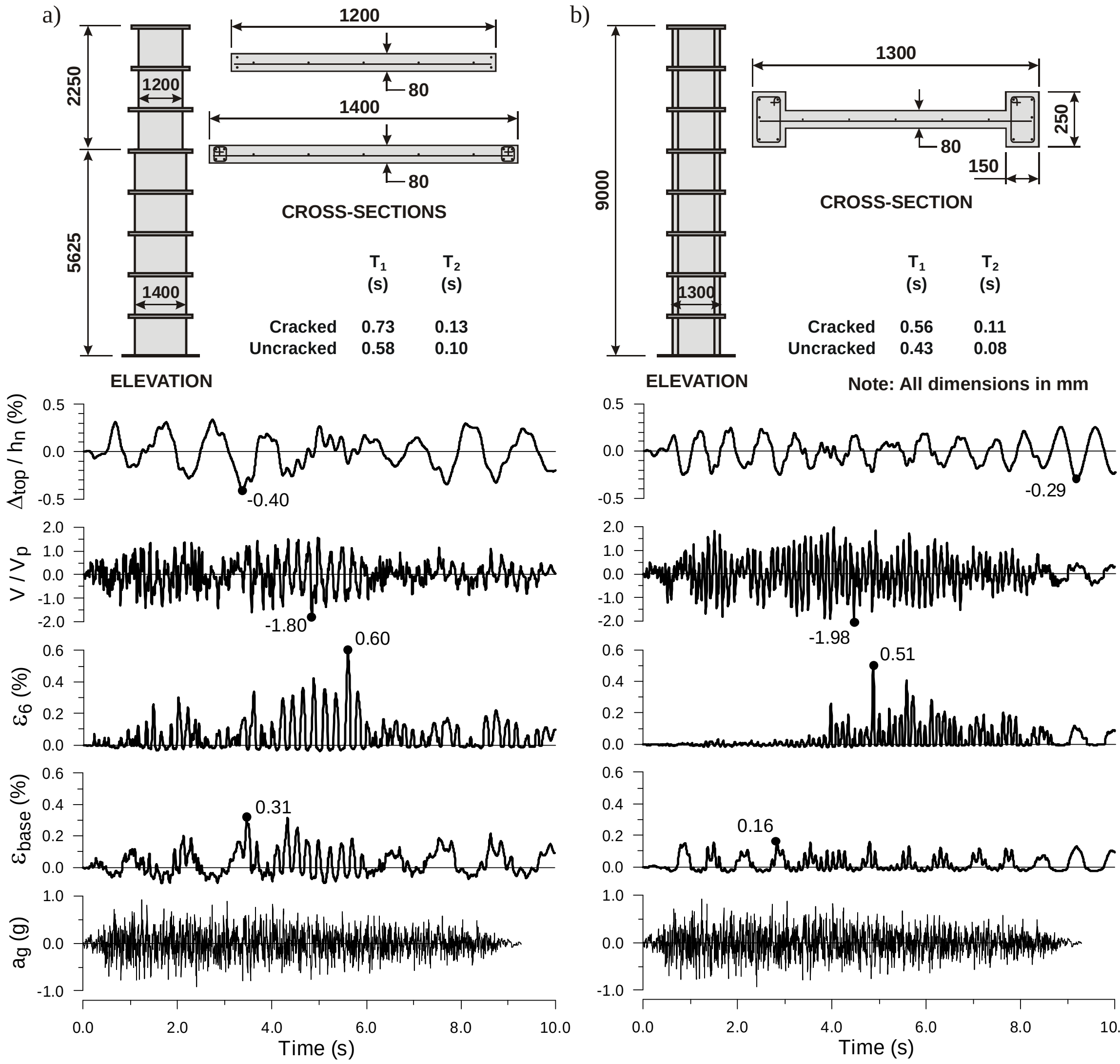

Figure 6 Properties and predicted response of test specimens studied: a) Rectangular wall; b) I-Shaped wall.

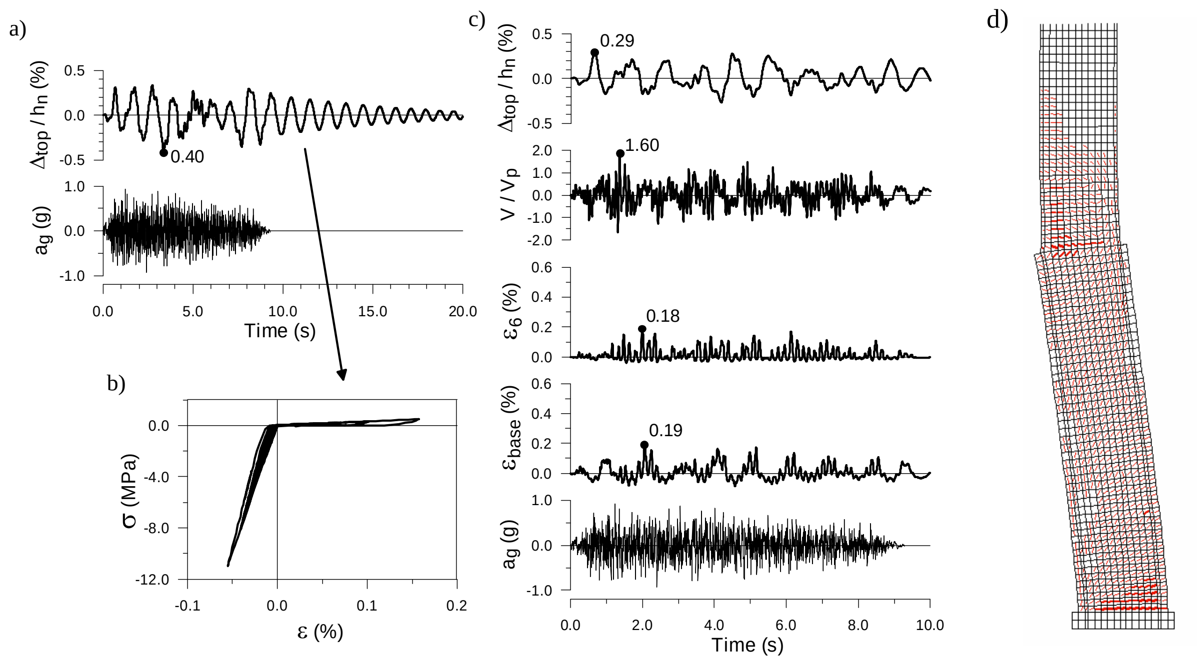

Figure 7 Additional analysis results for the rectangular wall: a) Decaying response after application of the ground motion; b) Hysteretic response of concrete under free vibration response; c) Response with 1.5% Rayleigh damping; and d) Crack pattern under the design ground motion amplified by 1.5.

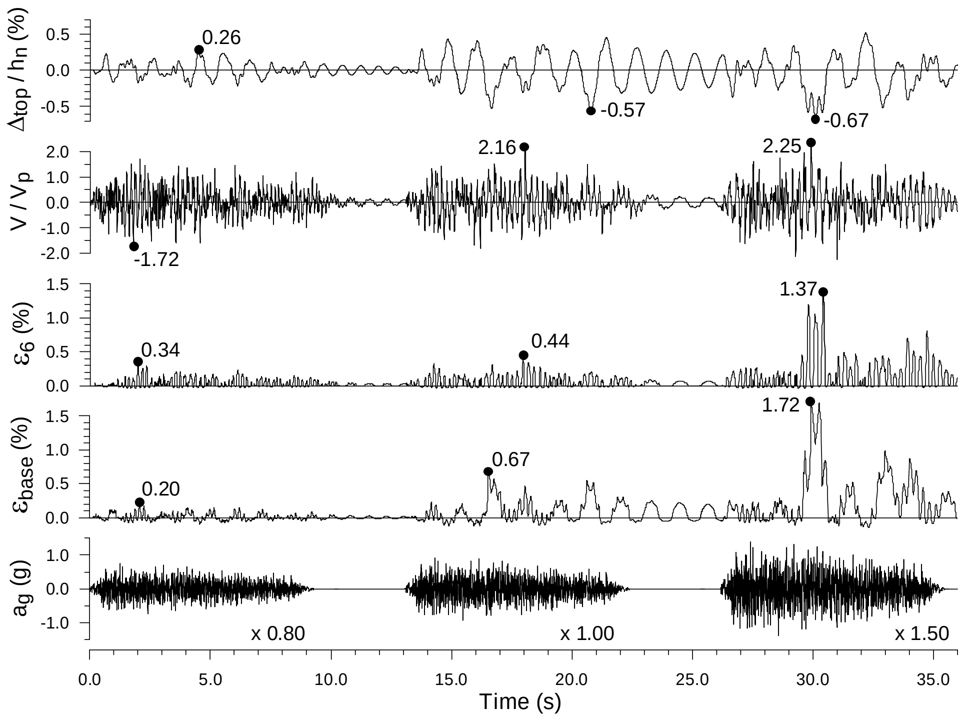

Figure 8 Predicted response of the rectangular wall specimen under successive application of ground motion amplitudes.

Figure 7c presents the response of the rectangular wall obtained when specifying 1.5% Rayleigh damping in the first two modes of vibration. The impact is significant: the roof drift and the strain demand at the 6th level reduce respectively from 0.40% and 0.60% to 0.29% and 0.18%. The base shear demand is also reduced, which illustrates the sensitivity of higher mode response to damping modelling assumptions. In the test program, it is planned to conduct successive tests at increasing amplitude of ground motions up to 150% of the design level. The crack pattern under 1.5 times the deign ground motion is shown in Fig. 7d. Significant inelastic rotation demand is expected at the base as well as at the 6th floor. Figure 8 shows the response of the rectangular wall under successive applications of the seismic excitation. When comparing the results under 1.0 times the design ground motion with the corresponding results in Fig. 6a, it is observed that the initial damage conditions can also impact significantly on the wall response.

4. CONCLUSIONS

The analysis of a 15-storey shear wall structures illustrated higher mode response on shear and flexural demand for reinforced concrete shear wall structures located at two different sites in Canada. The capability of reproducing the inelastic flexural and shear response of shear walls using reduced-scale specimens was verified experimentally. The test results could also be reproduced accurately using detailed FE simulations. Shake table test specimens were designed and were validated using FE analysis. Higher mode response is expected in the test models but the behaviour is found to be sensitive to the modeling of damping and the sequence of testing.

ACKNOWLEDGEMENTS

Funding for this research was provided by the Government of Quebec (FQRNT program) and the Natural Sciences and Engineering Research Council of Canada (Canada Research Chair Program).

REFERENCES

Boivin, Y. and Paultre, P. (2008). Seismic Performance of a 12-Storey Ductile Concrete Shear Wall System Designed According to the New Canadian Building Design Codes. *Proc. CSCE 2008 Annual Conf.*, Quebec, QC, Paper No. ST-442.

CSA. (2004). Design of concrete structures. Standard CAN/CSA-A23.3-04, Canadian Standard Association, Rexdale, ON.

Filiatrault, A., D'Aronco, D. and Tinawi, R. (1994). Seismic Shear Demand of Ductile Cantilever Walls: a Canadian Code Perspective, *Can. J. of Civ. Eng.*, **21:3**, 363-376.

Ghorbanirenani, I., Velev, N., Tremblay, R., Palermo, D., and Massicotte, B., and Léger, P. (2008). Modeling and Testing of Influence of loading history and scaling effects on the inelastic response of reinforced concrete shear walls. *ACI Structural J*. (submitted)

NRCC. (2005). National building code of Canada. 12th ed., National Research Council of Canada, Ottawa, ON.

Palermo, D., and Vecchio, F. J. (2003). Compression Field Modeling of Reinforced Concrete Subjected to Reversed Loading: Formulation. *ACI Struct. J.*, **100:5**, 616-625.

Panneton, M., Léger, P., and Tremblay, R. (2006). Inelastic Analysis of a Reinforced Concrete Shear Wall Building According to the NBCC 2005. *Can. J. of Civ. Eng.*, **33:7**, 854-871.

Priestley, M.J.N., and Amaris, A.D. (2002). Dynamic amplification of seismic moments and shear forces in cantilever walls. *Research Report ROSE-2002/01*, Rose School, University of Pavia, Pavia, Italy.

Sullivan, T.J., Priestley, M.J.N. and Calvi, G.M. (2006). Shear Forces in RC Walls of Frame-Wall Structures. *Proc. 8th U.S. Nat. Conf. on Earthquake Eng.* San Francisco, CA. Paper No. 1188.

Tremblay, R., Velev, N., Merzouq, S., Blais, C., Leclerc, M., Léger, P., Massicotte, B., and Rogers, C. (2005). Multi-purpose earthquake simulation testing set-up for seismic force resisting systems of multi-story buildings. *Proc. First Int. Conf. on Advances in Experimental Structural Eng.*, Nagoya, Japan, Paper No. 533.

Tremblay, R., Léger, P., and Tu, J. (2001). An Evaluation of P-delta Effects on the Inelastic Seismic Response of Concrete Flexural Walls. *Can. J. of Civ. Eng.*, **28:4**, 640-655.

Vecchio, F.J. (2000). Disturbed Stress Field Model for Reinforced Concrete Formulation. ASCE J. of Struct. Eng. **126:9**, 1070-1077.

Wong, P.S., and Vecchio, F.J. (2002). VecTor2 & Formworks User's Manuals. Dept. of Civ. Eng., Univ. of Toronto, Toronto, ON, Canada.