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SEISMIC SHEAR DEMAND IN WALL PIERS OF COUPLED AND PARTIALLY COUPLED SYSTEMS

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ABSTRACT: In absence of a reliable assessment of seismic shear demand using a linear analysis, the Canadian standard for the design of reinforced concrete structure (CSA A23.3) specifies a minimum shear capacity to avoid any weakness in shear of the concrete sections. However, this requirement do not seem sufficient for shear walls (SW) of the coupled systems. To this end, this paper presents results of analytical investigations conducted in order to get some insight into the shear demand in wall piers of coupled shear walls (CSW) and to improve thereby our understanding on the seismic behavior of CSW. The study considers a 20-storey height building with CSW having a degree of coupling DC=0.7 and with partially coupled systems with DC = 0.6. The building is founded on a category C soil and located in the cities of Montreal and Vancouver. The CSW are designed according to the seismic provisions of the National Building Code of Canada (NBCC 2010) and CSA A23.3-2009 and then subjected to nonlinear dynamic analyses. Results revealed that the design shear capacity specified by the standard may not be conservative. This undesirable behavior is mainly attributed to the inelastic effects of higher modes.

Key words: shear demand, coupled shear wall systems, higher modes, nonlinear analysis.

1. Introduction

Isolated and CSW are often used to provide lateral seismic resistance to multi-storey buildings. This is due to their in-plane stiffness which provides efficient lateral stability and the high ductile and energy absorbing capacity of their coupling beams (CBs).

The CBs of CSW designed according to the seismic requirements of NBCC and CSA A23.3 act as a primary energy dissipation system through formation of plastic hinges at the CBs ends before SW base. A force modification factor (R_d) to take into account the ductile capacity of structural system can be used for linear analysis. The design shear force thus obtained has to be increased to reach V_p, the probable shear force associated to the probable moment resistance of the concrete section considered in order to avoid premature brittle shear failure.

However, research studies have shown that, using this approach, the shear demand can be significantly underestimated at the base of ductile isolated SW (Filiatrault & al., 1994; Tremblay & al., 2001; White and Ventura 2004; Rad and Adebar 2009) and at SW base of coupled systems (Chaallal and Gauthier, 2000; Boivin and Paultre 2010). This is particularly true for isolated SWs and CSWs which have developed plastic hinges at their base. Indeed, in these sections, shear demand continues to grow with the seismic acceleration to reach and even exceed V_p, the minimum design capacity specified by the standard. This undesirable behavior is mainly attributed to the inelastic effects of higher modes (Tremblay & al., 2008; Ghorbanirenani, 2010). A dynamic amplification factor to account for these effects has been introduced recently in the CSA A23.3-2014, but only for isolated SWs, CSWs being excluded (see clause 21.5.2.2.7 of CSA A23.3 2014).

This has been the main impetus to conduct the present study in order to improve our understanding of the seismic behavior of CSWs. The objective of the study was to evaluate the shear demand at the base of SWs of coupled systems designed according to the Canadian seismic provisions.

2. Description of the analytical model

The CSWs considered in this study are part of a seismic resistant system of 20-storey office buildings located in two different sites, Montreal and Vancouver, representative of the seismic hazard of Eastern and Western Canada. Soil category C, corresponding to very dense soil or soft rock, and two degrees of coupling, DC=0.7 and DC=0.6, for coupled and partially coupled systems, are considered for both sites.

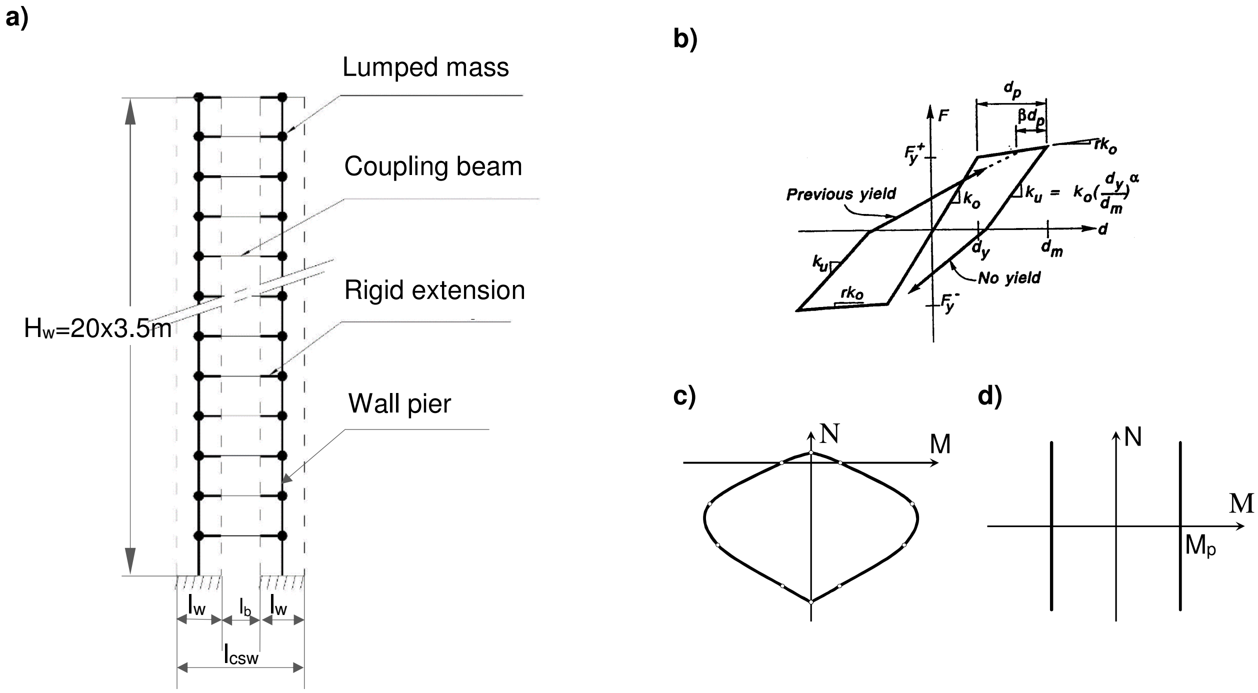

Fig. 1 – Modeling: a) Coupled shear walls; b) Modified Takeda degrading stiffness (S. Otani, 1974); c) Beam-column model; d) Giberson beam model

For linear and nonlinear numerical investigations, the coupled systems are represented by moment-resisting frames consisting of two large columns (SW) connected at each level by the CBs (see Fig. 1a). The extremities of the CBs are modeled by rigid extensions to simulate the large flexural stiffness of the wall piers on storey height. At each level, a lumped mass is considered at the intersection of the reference axis of the wall with that of the CB.

The CBs and wall piers of the coupled systems are designed and proportioned according to the seismic Canadian provisions (NBCC-2010 and CSA A23.3-2009/2014) using capacity design principle. This principle favors the development of the maximum ductile capacity of the CSWs by enforcing a desired hierarchy in the formation of plastic hinging, i.e., at the CBs ends before at SW base. This hierarchy is attained by imposing at each level a flexural capacity of wall pier greater than demand associated to the development of nominal flexural capacity in the CBs above. In addition, this same principle applied locally at the concrete section level by favouring the development of full flexural capacity before that in shear to avoid premature brittle failure. However, shear capacity greater than the probable demand, V_p, is to be maintained over the plastic hinge height at the SW base of the coupled systems. Above the plastic hinge, the shear demand from analysis must be increased accordingly by the ratio V_p/V_f calculated at the upper limit of the plastic hinge zone, where V_f is the factored shear demand. Dynamic (response spectrum) analysis method is used to determine the bending moments and the shear forces in the CBs and the wall piers of coupled systems analyzed. These coupled and partially coupled systems are designed as ductile, i.e., with a ductility-related force modification factor R_d equal to 4.0 and 3.5, respectively.

3. Nonlinear analysis - Presentation and Interpretation of results

The CSWs are modeled as moment-resisting frames using beam-column model for the wall piers and Giberson beam model for the CBs (see Fig. 1) using the Ruaumoko computer program (Carr 2006). These beam elements are able to develop plastic hinges at their ends during the nonlinear analysis. The modified Takeda hysteresis model is adopted for the plastic hinges (S. Otani, 1974). Only 2D analysis is performed herein and to account for torsional effect the seismic signals are increased by 25%.

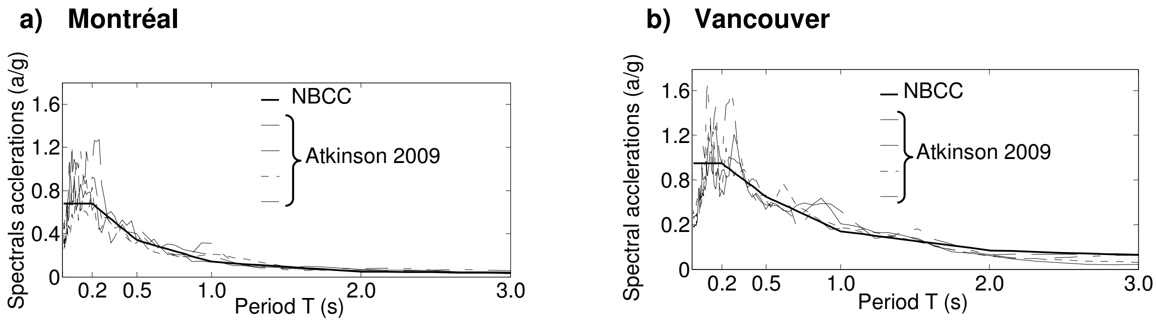

Fig. 2 – Seismic signals (Atkinson 2009) calibrated compatible to acceleration spectrum

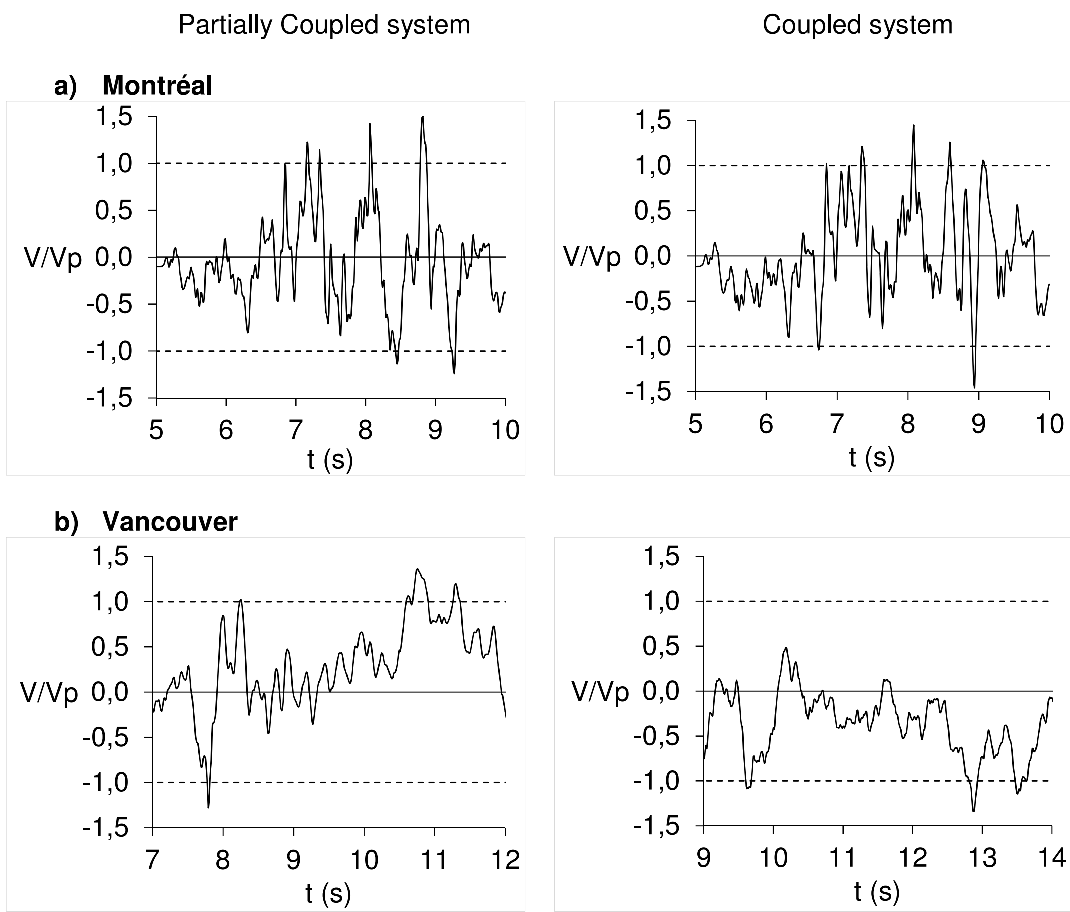

Fig. 3 – Time history of shear seismic response at the SW base of the 20 storey coupled system

Two sets of four seismic signals were considered for the nonlinear analysis of the coupled systems. They were calibrated compatible to the design spectra of each site. The first set consists of the four seismic synthetic signals from Beresnev and Atkinson (1998), M7.0R70 for Montreal and M7.2R70 for Vancouver.

The second set is made of the four best-rated seismic synthetic signals selected for each site in Atkinson's catalog (2009). The best-rated seismic signal is the one with an average spectral distance between 0.5 and 2 and that has the acceleration spectrum the most "parallel" to the targeted design spectra. The calibration for the first set is multi-factor in conformity with the approach presented in Benazza and Chaallal (2010), whereas for the second set it is a single-factor calibration (the ratio of average spectral distance).

Figure 3 presents the computed time history response for both sites. It shows selected windows around the peak of the shear demand at SW base of the 20-storey coupled and partially coupled systems analysed. The figure clearly shows that base shear response exceeds several times the design value V_p but also that the responses are dominated by the higher modes, particularly for Montreal, which is characterized by seismic signals rich in high frequencies.

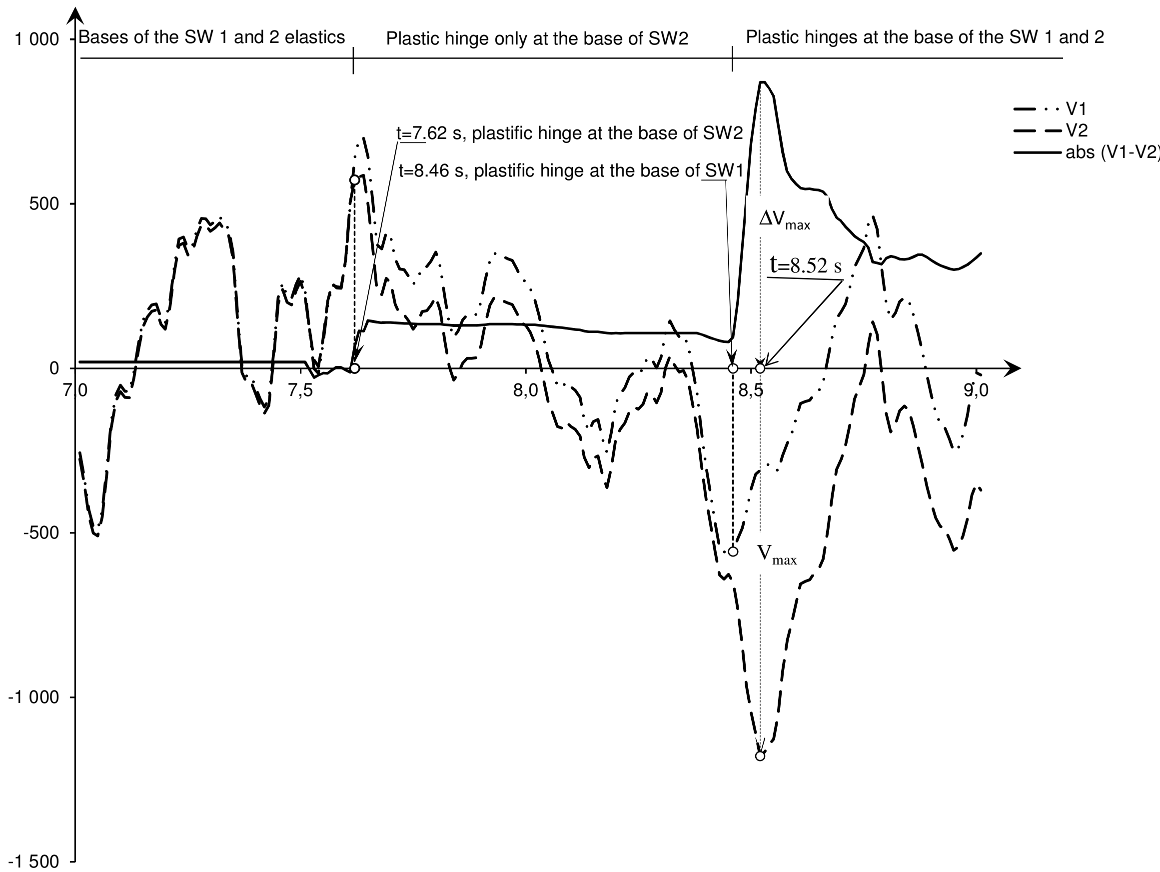

Fig. 4 – Typical evolution of the distribution of shear demand between SW1 and SW2 (kN)

The overall shear demand at the SW base of coupled systems evolves with the intensity of the seismic acceleration during the loading cycles. The distribution of the shear demand between the concrete sections at the SW base mainly depends on the curvature demand in each of these two elements. As long as curvature demand remains elastic, the distribution is 50%-50% (because the coupled system is symmetric). Otherwise, it depends on the curvature demand reached at each instant in adjacent concrete sections and can therefore be greatly disproportionate (25%-75% or more) (see Figures 4 and 5).

Figure 4 illustrates the evolution of shear demand of CSWs and its distribution between SW1 and SW2 of the coupled systems during seismic loading. It reveals the complexity of the shear behavior of this structural system. Clearly, the shear demand at the SW base of coupled systems depends not only on the intensity of the overall demand but also on the curvature ductility demand in the adjacent concrete sections. The shear demand has been at several instances greater than the design shear capacity. For example, the excess of demand with respect to the design capacity has reached 50% for partially coupled system in Montreal.

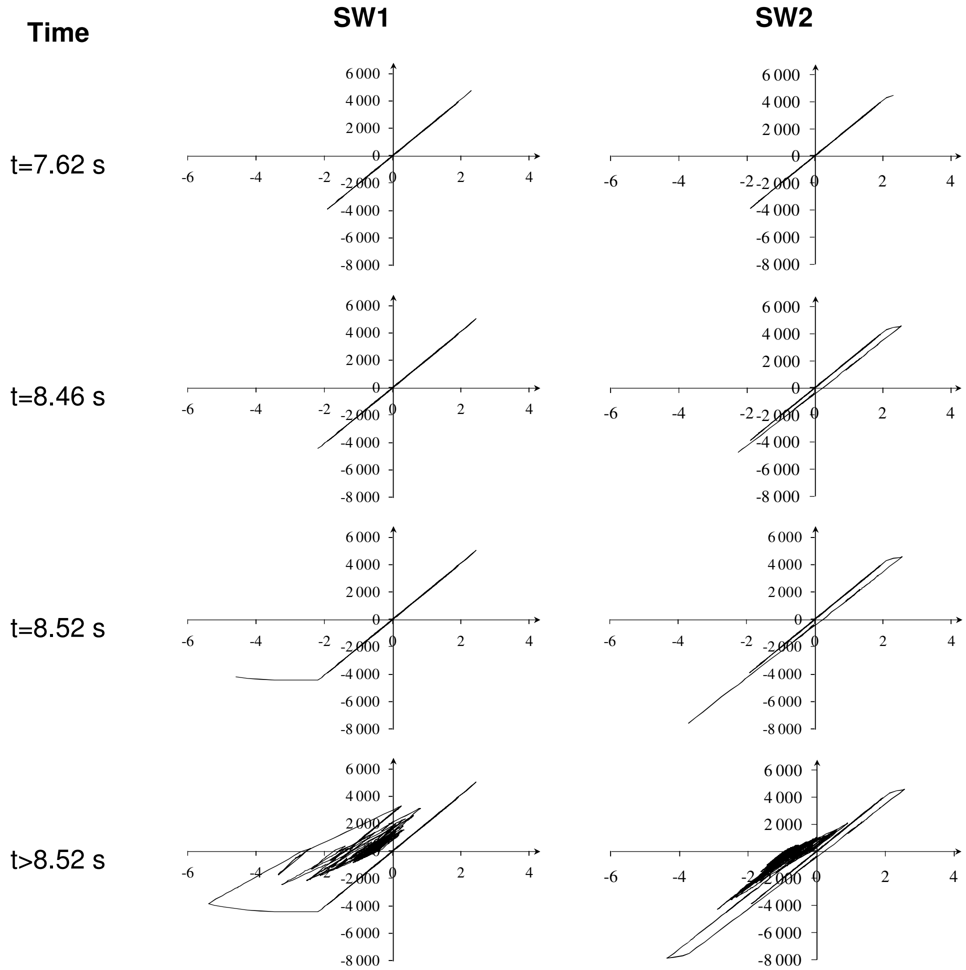

Fig. 5 – Evolution of ductility demand at SW base of coupled systems during loading cycles

4. Conclusions

The results of this study show that the shear demand can be several times greater than the minimum design capacity specified by the Canadian standards, V_p (see Figure 3). This significant shear demand at the SW base of coupled systems is partly due to the redistribution of overall demand in the inelastic response phase (see Figure 4 and Table 1). The excess of shear demand has reached nearly 50% of V_p in the city of Montreal for the partially coupled system. This system (DC = 0.6) more rigid than the coupled system (DC = 0.7) attracts more shear. In addition, Montreal is located in eastern Canada characterized by seismic signals rich in high frequencies contributing to obtained responses associated to higher modes. Finally, a dynamic amplification factor (of 1 to 1.5) is suggested for these structural systems to account for this excess of demand due to the inelastic effect of higher modes.

The findings of this study were obtained for the conditions and shear wall systems considered herein. Therefore, to be generalized, they should be validated by further studies that should consider a larger spectrum of parameters. The fundamental mode period of vibration, the frequency content characterizing the seismic zone and the wall overstrength factor are among important parameters to be encompassed.

5. Acknowledgements

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