
# Influence of the Degree of Coupling on Shear Amplifications due to Higher Modes Effects in RC Partially Coupled Wall Structures

Gabriel Rivard[1], Steeve Ambroise[2], Patrick Paultre[3]

[1] Master Student, Department of Civil and Building Engineering, University of Sherbrooke - Sherbrooke, QC, Canada.
[2] Research professional, Department of Civil and Building Engineering, University of Sherbrooke - Sherbrooke, QC, Canada.
[3] Professor, Department of Civil and Building Engineering, University of Sherbrooke - Sherbrooke, QC, Canada.

## ABSTRACT

This study evaluates the impact of the degree of coupling of partially coupled walls structures on the seismic shear forces amplification due to nonlinear higher mode effects. The parameters varied include, among others, the fundamental period of the structure, the flexural overstrength of the walls at the base, and the ground motion characteristics representative of eastern and western Canada earthquakes. The concluding results drawn from this work are based on more than 10,000 nonlinear time-history analyses, where the ground motion time series were selected based on the conditional spectrum record selection algorithm. It is found that the seismic shear force amplification decreases with increasing degree of couplings, and that the nonlinear higher mode effects are more critical when the fundamental period is larger than 1 s.

Keywords: shear amplification, coupled walls, degree of coupling, wall flexural overstrength, nonlinear time-history analysis.

## INTRODUCTION

The seismic force distribution on reinforced concrete shear wall structures is generally dominated by higher lateral modes of vibration, unlike the displacement distribution which is usually controlled by the fundamental lateral mode of vibration of the structures. The predominating contribution of higher lateral modes in the elastic range produces moment and shear force distributions significantly different from and larger than those resulting from the static code procedure which is traditionally based on the fundamental lateral mode of vibration. An additional dynamic effect occurs when the wall response changes from elastic to inelastic behaviour, giving rise to an increase in the relative contribution of higher lateral modes [1, 2].

To ensure that inelastic mechanisms develop as intended, and no undesirable failure modes occur, the identified plastic hinge zones of the seismic force resisting system (SFRS) of a building are specially designed and detailed to possess enough flexural ductility. This capacity design approach implemented in the Canadian Standards Association (CSA) standard A23.3 [3] also guarantees adequate shear strength to ensure a flexure-governed inelastic lateral response of the SFRS.

Shear amplification due to nonlinear higher mode effects are now well recognized for cantilever wall structures and it has been the subject of several numerical studies [4-7]. Recent studies [8-10] have led to significant changes in the 2014 edition of CSA A23.3 which proposes a simple way to amplify shear forces to account for nonlinear higher mode effects. However, this factor applies only to isolated structural walls and there is currently no reference available that considers the shear amplification in coupled walls due to nonlinear higher mode effects. The capacity design procedure of coupled walls, which are an assembly of at least two walls connected with coupling beams, enforces the formation of plastic hinge at the base of the walls after yielding of the beam's reinforcement on each floor.

The main objective of the current research is to investigate, through a numerical parametric study, the influence of the degree of coupling, among other parameters, on the nonlinear higher mode shear force amplification in partially coupled walls in which the degree of coupling (DOC), i.e., the portion of base overturning moment resistance provided by axial forces in wall piers resulting from shear in coupling beams is less than 66 %. In this study, the partially coupled walls, are designed and detailed according to the 2015 NBCC Code and the CSA A23.3-14 Standard.

## CONSIDERED METHODS OF ANALYSIS

Among the numerous parameters that may influence the seismic shear amplification in coupled walls in a capacity-design-based approach, this study concentrates on the impact of the degree of coupling. A parametric study was performed to identify and quantify the influence of the DOC parameter on the shear demand of buildings whose SFRS is made of coupled walls. The methodology selected for this analysis involves:

1. Selecting the ground motion records representative of eastern and western Canada;
2. Designing the wall pier and coupling beam elements for each case;
3. Estimating the shear demand with nonlinear time-history analyses (NLTHA).

The parameters considered in this study are:

- Degree of coupling (DOC);
- Fundamental period of vibration;
- Flexural overstrength at the base; and
- Ground motion selection.

As the two main regions of Canada (eastern and western) present different seismic frequency content, a representative city for each region will be designated for the study. For western Canada, the city of Vancouver was selected since it is the Canadian city with highest seismic risk. Likewise, for eastern Canada, the city of Baie-Saint-Paul was selected given its high frequency content, characteristic of cities in eastern Canada.

**Degree of coupling**

The DOC of a coupled wall system is defined as the ratio of the total overturning moment resisted by the coupling action with respect to the total overturning moment at the base resisted by traction and compression in walls under the action of an inverted triangular load:

$$\mathbf{DOC} = \frac{\boldsymbol{P_f\, l_{cg}}}{\sum \boldsymbol{M_w} + \boldsymbol{P_f\, l_{cg}}} \tag{1}$$

where $P_f$ is the axial forces (tension or compression) in walls due to shear forces in the coupling beams acting at the centroid of the coupled walls; $l_{cg}$ is the lever arm between centroids of the wall piers; and $M_w$ are the overturning moments at each wall piers (see Figure 1).

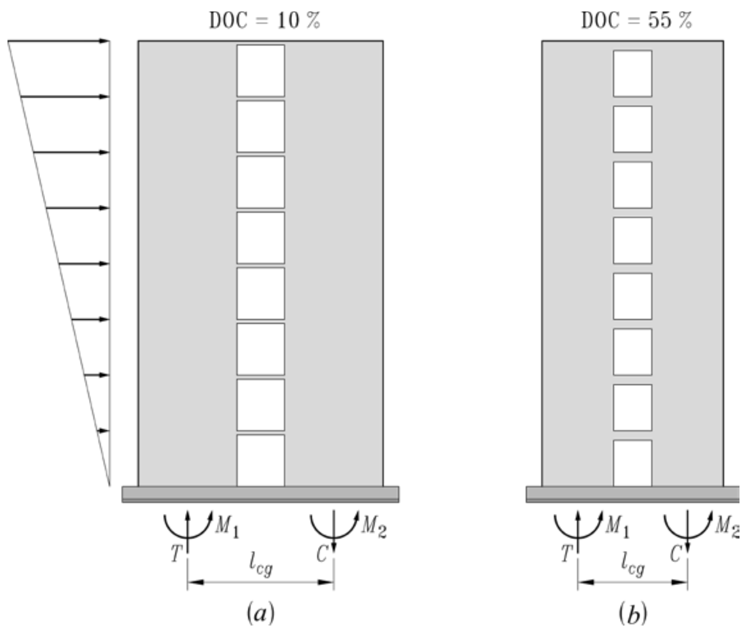


*Figure 1. Geometry of wall systems (a) weakly coupled, (b) moderately coupled.*

In this study, the DOC is varied by modifying the slenderness of the walls and the coupling beams. Very flexible coupling beams result in the coupled wall system to act as two isolated walls and, as the stiffness of the coupling beams increases, the walls system behaves like a single perforated wall with limited frame action. The 2015 NBCC [11] defines two types of coupled walls based on the DOC: partially coupled walls have a DOC less than 66 % while coupled walls are those that have a

DOC equal or larger than 66 %. Only partially coupled walls are considered in this investigation, with DOC equals to 30, 40, 50 and 60 %.

**Fundamental period of vibration**

The fundamental period corresponds to the first elastic period of numerical models according to the requirements of the 2015 NBCC and the CSA A23.3-14. In the studied models, section properties were reduced in order to consider concrete cracking. The geometric dimensions of the models were selected so that the fundamental period was between once and twice the empirical period recommended by the 2015 NBCC for structural wall systems:

$$\boldsymbol{T_a = 0.05 h_n^{3/4}} \tag{2}$$

where $h_n$ is the height of the structure.

**Flexural overstrength at the base**

Wall flexural overstrength at the base ($\gamma_w$) is defined as the ratio of the nominal flexural resistance ($M_n$) to the factored design moment ($M_f$) caused by seismic loading, and it is controlled by walls and coupling beams steel reinforcement. This factor is used in the CSA A23.3-14 Standard to determine the effective properties of concrete walls and as an indicator of the reserve flexural capacity compared to the strength required to withstand design forces. In this study, the target $\gamma_w$ values range from 2.0 to 4.0.

**Ground motion selection**

A large number of acceleration time histories needs to be used for NLTHA. It has been shown that selecting earthquake motions scaled about the uniform hazard spectra (UHS) is too conservative [12, 13, 14] because the UHS is defined for earthquake hazards having the same probability of exceedance for all periods. Instead, it has been proposed to use an approach for record selection based on the conditional mean spectrum (CMS) leading to selected ground motions that are more representative and consistent for a given seismic intensity measure (IM) [14]. The ground motions compatible with the CMS consider the variability and uncertainty of earthquakes and are more representative of real seismic events. Generally, the spectral acceleration at the fundamental period is used as the conditioning seismic IM for building the CMS. Dezhdar, E. and Adebar, P. [15] have shown that this alternative was not always adequate depending on the response sought. In fact, maximum shear response will be observed if the second period of vibration is selected as the conditioning period since shear response is more influenced by higher modes. Thus, for maximizing the shear response of 10-storey structures and more, the second elastic period of vibration is used as the conditioning period, whereas the fundamental period was used for structures with less than 10 stories.

A prescreening of the database in terms of magnitude, distance to the source, and soil type was performed to limit the selection of ground motions to those that closely describe the seismic characteristics at the site, followed by spectral matching to a target spectrum. The buildings are designed considering a site class with stiff soil (type D according to the 2015 NBCC), for which the average shear wave velocity in the first 30 meters is between 180 m/s and 360 m/s.

The spectral responses ($S_a$) at the target periods were chosen with a 2 % probability of exceedance over a return period of 50 years (return period of 2475 years). The identification of the seismic scenario in terms of mean magnitude ($M$) and epicentral distance ($R$) has been done by performing a disaggregation of the seismic hazard with the open-source software OpenQuake [16]. The mean spectrum $\left(\mu_{\ln S_a}\right)$ and the standard deviation $\left(\sigma_{\ln S_a}\right)$ were obtained using the appropriate ground motion prediction equations (GMPE).

For each selected conditioning period, a set of about forty ground motions corresponding to shallow crustal earthquakes were selected and scaled. The PEER NGA-West2 database [17] was used for both regions due to the limited number of high-intensity records for eastern Canada. Studies conducted by Baker [12] have shown that these accelerograms can produce an adequate estimation of the SFRS response if the shape of their spectrum is similar to the target spectrum.

**Seismic design and detailing**

To ensure the safety of reinforced concrete structural walls, the design envelope for shear force must be greater than the demand over the entire height of the SRFS. According to the CSA A23.3-14 Standard, this envelope is determined by taking the minimum of:

- the shear that corresponds to the development of the probable flexural resistance $\left(M_p\right)$ at the base. This envelope is obtained by multiplying the factored shear envelope $\left(V_f\right)$ by the probable overstrength factor $\left(\gamma_p\right)$, calculated at the base;

$$V_p = \gamma_p V_f = \left(\frac{M_p}{M_f}\right)_{\text{base}} V_f \tag{4}$$

- the shear determined from the load combination considering the effects of the earthquake using the seismic force reduction factors $R_d R_o = 1.3$, where $R_d$ is the seismic force reduction factor related to the ductility, and $R_o$ is the overstrength-related reduction factor.

The probable overstrength ($\gamma_p$) is calculated by taking an effective yield stress of $1.25\mathrm{f_y}$ for the reinforcement of walls and coupling beams. To estimate the $\gamma_p$ at the base of partially coupled walls, for which the two walls are not subject to significantly different axial forces caused by gravity load, the considered approach is based on the following assumptions:

- Each wall must be designed to withstand the maximum shear force to which it will be subjected to during an earthquake.
- Shear demand in the walls is proportional to the bending moment acting at the base of each wall rather than the total overturning moment acting on the coupled walls system.

This approach leads to two design envelopes, one for the tension wall (the wall with the larger algebraic axial force, considering tension is positive) and one for the compression wall (the wall with the lower algebraic axial force).

**Modeling for NLTHA**

Planar coupled walls were used in this study as SFRS as shown in Figure 2. NLTHA were performed using the Open System for Earthquake Engineering Simulation (OpenSEES) [18]. The walls were modelled using force-based multilayer beam-column elements and the mesh included one element per story with 5 integration points as shown in Fig. 2.

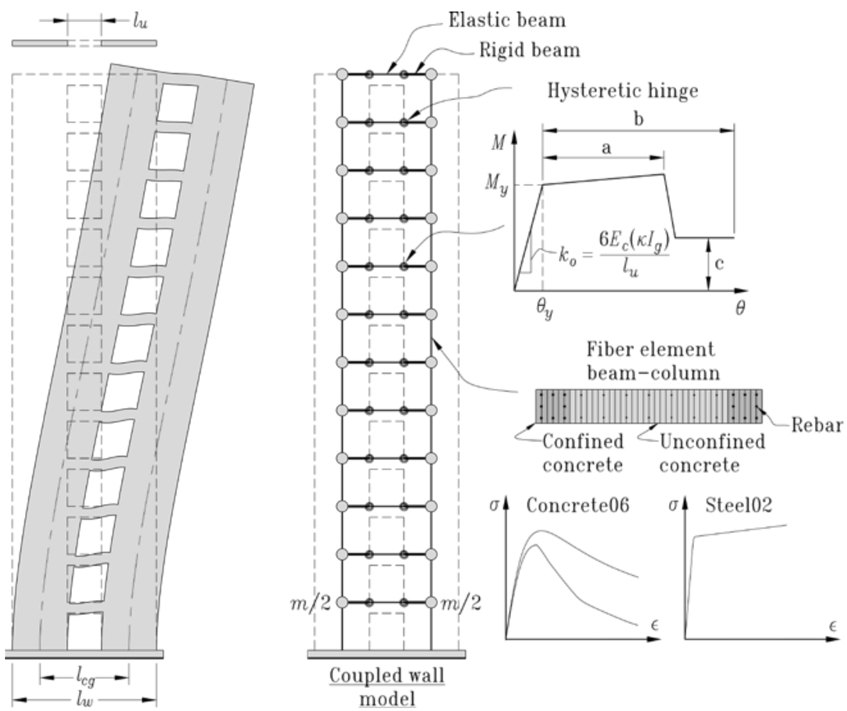


*Figure 2. Multilayer beam-column elements on OpenSEES.*

The uniaxial concrete behaviour was modelled with OpenSEES *Concrete06* material law which is based on the model proposed by Popovics [19] while the reinforcement was modelled with the *Steel02* material law which considers strain hardening and Bauschinger effects. The fiber elements do not allow to consider the interaction between shear, flexural and axial forces. Thus, the shear deformations were considered uncoupled with an elastic behaviour. P-Delta effects were included through a corotational transformation. The mass of the building was concentrated at each floor.

The coupling beams were modelled with beam elements with concentrated plasticity at the ends of the beam in the form of nonlinear hinges with determined hysteretic behaviour. The initial stiffness of these hinges was calculated from an effective inertia with the expression proposed by Son Vu. et al. [20] for conventionally and diagonally reinforced coupling beams. The moment-rotation behaviour was described with the modified Ibarra-Medina-Krawinkler deterioration model with bilinear hysteretic response. The backbone curve parameters shown in Figure 2 were taken from the ASCE 41-13 [21] and are presented in Table 1. Rigid beams were used to connect the hysteretic hinges to the walls center of gravity while elastic beams were modelled between the two nonlinear hinges.

*Table 1. Nonlinear parameters for coupling beams modelling [ASCE 41-13].*

| **Type of coupling beam reinforcement** | **Chord rotation a** | **b** | **c** |
|---|---|---|---|
| Conventional | 0.02 | 0.04 | 0.50 |
| Diagonal | 0.03 | 0.05 | 0.80 |

A Rayleigh damping model, proportional to mass matrix and tangent stiffness matrix, with a damping ratio of 2 % at the first and the last lateral modes was considered in the NLTHA.

## CASE STUDIES

The list of studied cases for this parametric study are presented in Table 2. For western Canada, 118 models were built while only 88 were developed for the eastern region. Each of them was subjected to a series of 40 ground motions which resulted in a total of more than 8,000 NLTHA of partially coupled walls for both regions.

*Table 2. Partially coupled walls' parameters variation.*

| **Number of Stories, $N$** | **Fundamental Period, $T$** | **Degree of Coupling, DOC** | **Flexural Overstrength, $\gamma_w$** |
|---|---|---|---|
| 3 | 0.3 | 30, 40, 50, 60 | 2.0, 2.5, 3.0, 4.0 |
| 5 | 0.5 | 30, 40, 50, 60 | 2.0, 2.5, 3.0, 4.0 |
| 10 | 1.0 | 20, 30, 40, 50, 60 | 2.0†, 2.5, 3.0, 4.0 |
| 10 | 1.5 | 30, 40, 50, 60 | 2.0†, 2.5, 3.0, 4.0 |
| 15 | 2.0 | 30, 40, 50, 60 | 2.0†, 2.5, 3.0, 4.0 |
| 20 | 2.5 | 30, 40, 50, 60 | 2.0†, 2.5†, 3.0, 4.0 |
| 25 | 3.0 | 30, 40, 50, 60 | 2.0†, 2.5†, 3.0, 4.0 |

†: Data not available for Eastern Canada

Although the case of shear walls has been largely analyzed in the literature, these systems have also been studied as part of this project. A total of 28 models for western Canada and 27 models for eastern Canada were studied which led to more than 2 000 additional NLTHA. These walls can be considered as coupled walls with a $\mathrm{DOC} = 0$. The new $\bar{\omega}_v$ factor proposed by CSA A23.3-14 was not included in the design forces. The objective is to verify the previous changes and to compare results with those from partially coupled wall systems.

## RESULTS AND DISCUSSION

The seismic shear amplification factors at the base for each model are calculated from the average of the results obtained from 40 ground motion excitations selected with the CMS method (Figures 3-9). They represent the ratio between the maximum of the shear demand observed during the NLTHA to the design shear $(V_p)$ from equation (4).

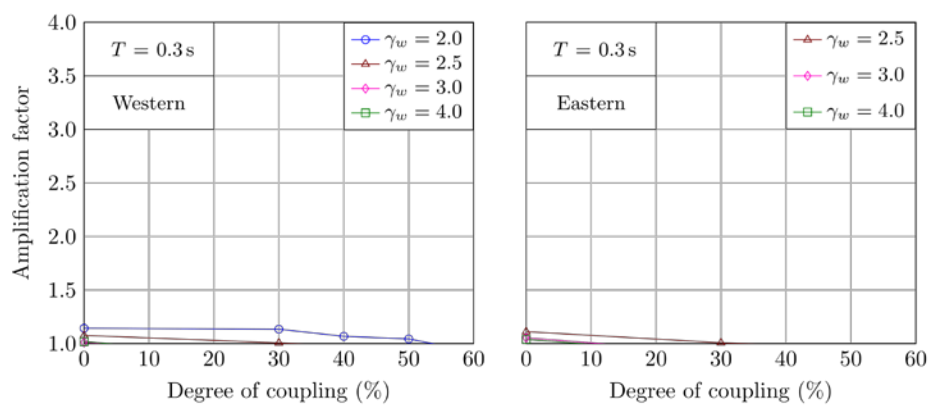


*Figure 3. Influence of the $DOC$ on seismic shear amplification factor, $T = 0.3\ s$.*

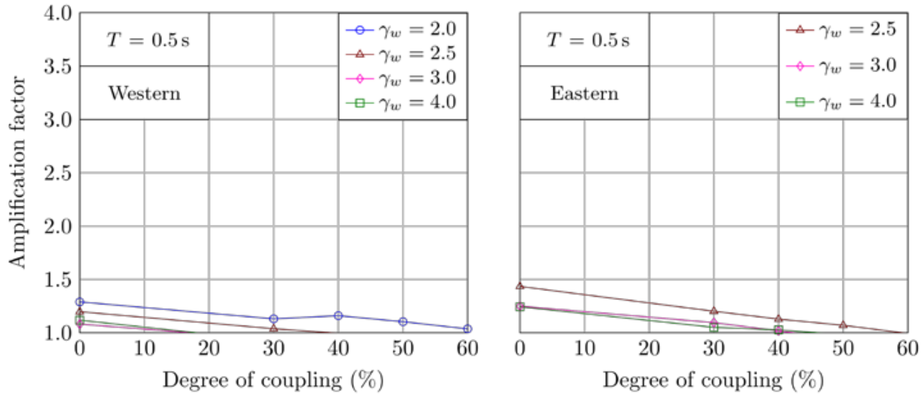


*Figure 4. Influence of the $DOC$ on seismic shear amplification factor, $T = 0.5\ s$.*

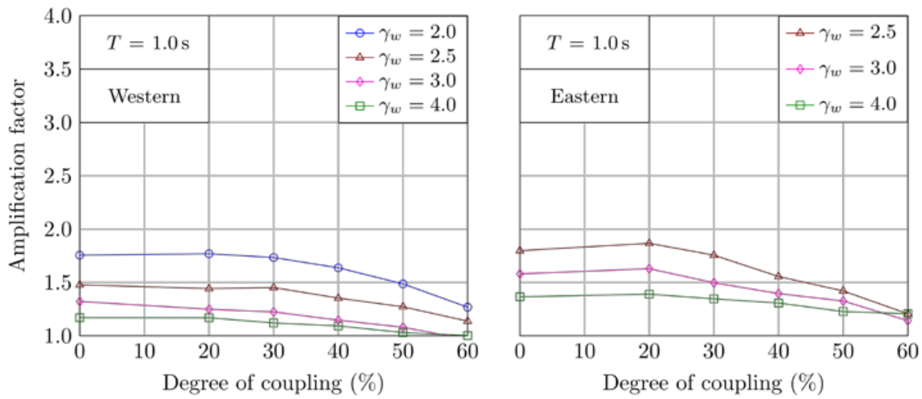


*Figure 5. Influence of $DOC$ on seismic shear amplification factor, $T = 1.0\ s$.*

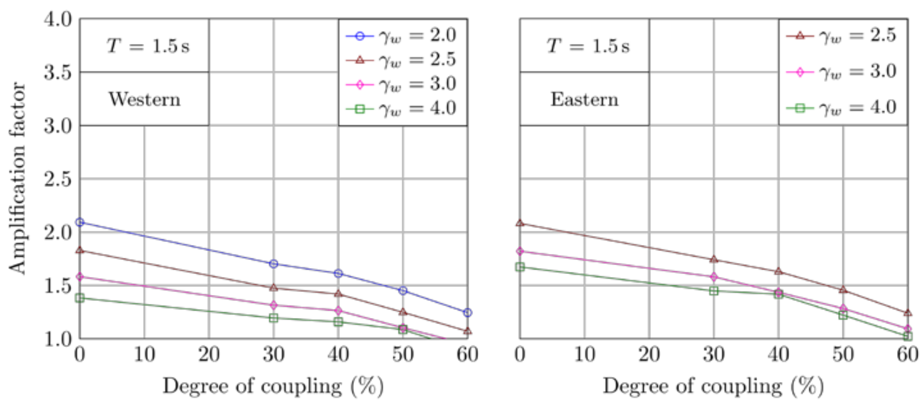


*Figure 6. Influence of the $DOC$ on seismic shear amplification factor, $T = 1.5\ s$.*

The results from these NLTHA show that the shear amplification decreases as the flexural overstrength at the base $\gamma_w$ increases. Given that design forces are greater when the flexural capacity is increased, this leads to a reduction in the shear amplification factor. It should also be noted that the rate of decrease is globally higher for lower $\gamma_w$ values.

The influence of the fundamental period of vibration in coupled walls is more significant when the period shifts from 0.5 s to 1.5 s whereas, for periods greater than 2.0 s, the results show an almost constant amplification factor for the two studied regions. Furthermore, the influence of the period $T$ is greater in eastern Canada which is characterized by high-frequency content earthquakes.

The results show a clear reduction of the dynamic amplification factor as the DOC increases. In partially coupled walls, there is no seismic amplification observed with a DOC close to 60 % while for systems with a low DOC, similar values to those for shear walls were found for $T \leq 1\ s$. The dynamic amplification decreases about linearly with the DOC when it goes from 30 % to 60 %.

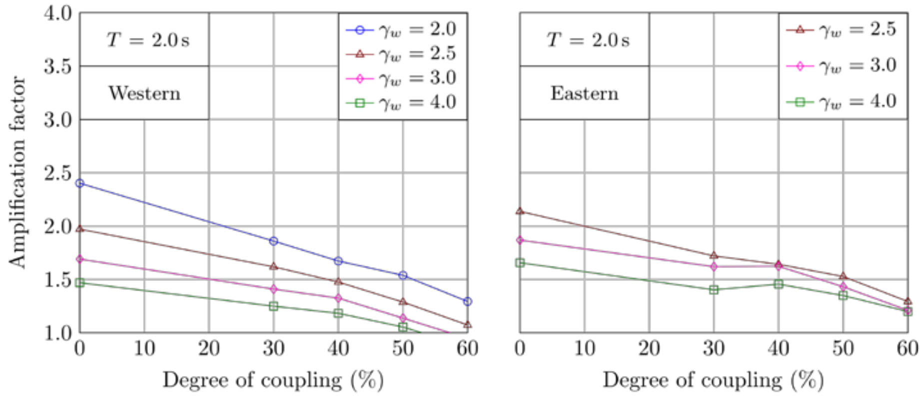


*Figure 7. Influence of the $DOC$ on seismic shear amplification factor, $T = 2.0\ s$.*

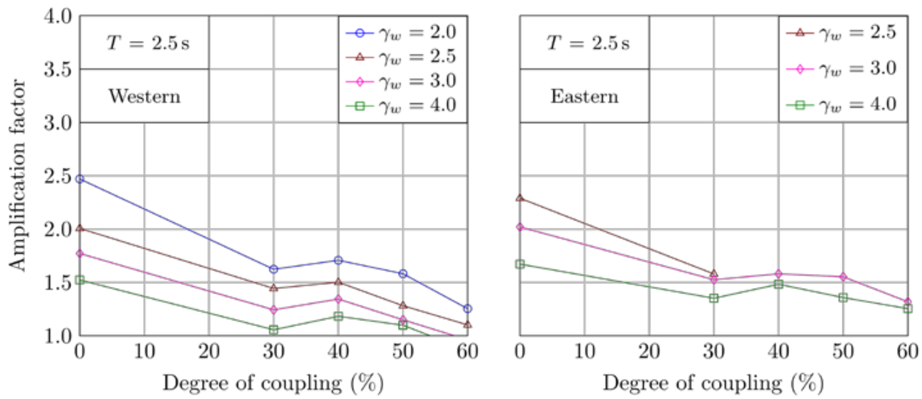


*Figure 8. Influence of the $DOC$ on seismic shear amplification factor, $T = 2.5\ s$.*

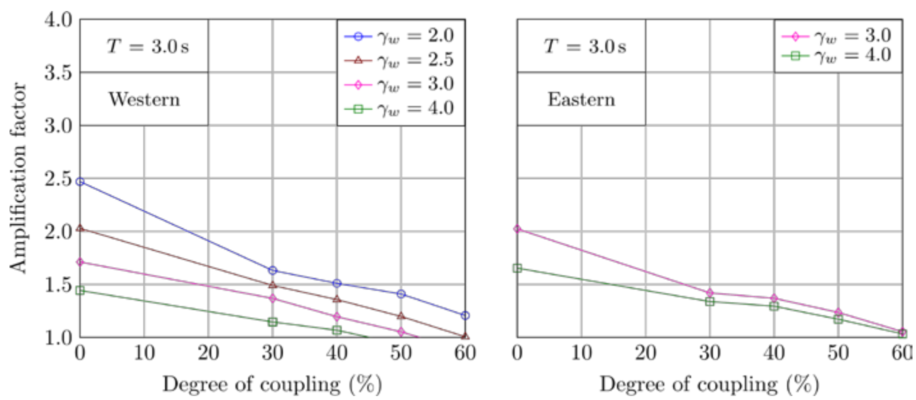


*Figure 9. Influence of the $DOC$ on seismic shear amplification factor, $T = 3.0\ s$.*

## CONCLUSIONS

This research focused on the seismic shear demand of partially coupled walls. Recent studies have shown lower values of amplification factor in coupled walls, which is owed to the presence of coupling beams. This parametric study has shown that the shear amplification due to nonlinear higher modes effects is significant in coupled walls and must, therefore, be considered. The main conclusions can be summarized as follows:

- The shear amplification factor is lower in structures with high flexural overstrength at the base, $\gamma_w$.
- The nonlinear higher mode effects on the shear forces are more significant in buildings whose fundamental period is greater than 1 s, and tend to remain constant for periods greater than 1.5 s.
- The shear amplification is more important for eastern Canada where earthquakes are characterized by a higher frequency content.
- The seismic amplification of seismic shear forces decreases with an increase of the degree of coupling.

**ACKNOWLEDGMENTS**

The authors gratefully acknowledge the financial support of the Natural Sciences, the Engineering Research Council of Canada (NSERC), the Fonds de Recherche du Quebec - Nature et Technologies (FRQNT) and the Centre d'études interuniversitaire des structures sous charges extrêmes (Interuniversity Centre for Studies of Structures under Extreme Loads).

**REFERENCES**

[1] Seneviratna, G.D.P.K., and Krawinkler, H. (1994). "Strength and displacement demands for seismic design of structural walls." In *the 5th US National Conference on Earthquake Engineering*, Chicago, Ill., 10–14 July 1994, Earthquake Engineering Research Institute (EERI), Oakland, Calif., Vol. II, 181–190.

[2] Priestley, M.J.N. (2003). "Does capacity design do the job? An examination of higher mode effects in cantilever walls." *Bulletin of the New Zealand Society for Earthquake Engineering*, 36(4), 276–292.

[3] Canadian Standard Association - CSA (2014). *CAN/CSA-A23.3: Calcul des ouvrages en béton armé*. Prepared by the CSA, Toronto, ON.

[4] Blakeley, R., Cooney, R., and Megget, L., (1975). "Seismic shear loading at flexural capacity in cantilever wall structures." *Bulletin of the New Zealand National Society for Earthquake Engineering*, 8(4) 278–290.

[5] Filiatrault, A., D'Aronco, D., and Tinawi, R., (1994). "Seismic shear demand of ductile cantilever walls: a Canadian code perspective." *Canadian Journal of Civil Engineering* 21, 363–376.

[6] Tremblay, R., Léger, P., and Tu, J., (2001). "Inelastic seismic response of concrete shear walls considering P-delta effects." *Canadian Journal of Civil Engineering* 28, 640–655.

[7] Sullivan, T.J., Priestley, M.J.N., and Calvi, G.M., (2008). "Estimating the Higher-Mode Response of Ductile Structures." *Journal of Earthquake Engineering* 12, 456–472.

[8] Dezhdar, E., (2012). *Seismic response of cantilever shear wall buildings*. Ph.D. thesis. Dept. of Civil Eng., The University of British Columbia, Vancouver, BC, Canada.

[9] Boivin, Y. and Paultre, P., (2012). "Seismic force demand on ductile reinforced concrete shear walls subjected to western North American ground motions: Part 1 - parametric study." *Canadian Journal of Civil Engineering*, 39, 723–737.

[10] Ambroise, S., Boivin, Y., and Paultre, P., (2013). "Parametric study on higher mode amplification effects in ductile RC cantilever walls designed for western and eastern Canada." In *CSCE 2013 General Conference*, DIS-81, May 29–June 1, Montreal, Qc, Canada.

[11] NBCC, (2015). *National Building Code of Canada - Part 4: Structural Design*. National Research Council Canada, 14 edn. Canadian Commission on Building and Fire Codes, Ottawa, Ont.

[12] Baker, J. W. and Allin Cornell, C., (2006). "Spectral shape, epsilon and record selection." *Earthquake Engineering & Structural Dynamics*, 35, 1077–1095.

[13] Baker, J. W., (2011). "Conditional Mean Spectrum: Tool for Ground-Motion Selection." *Journal of Structural Engineering*, 137, 322–331.

[14] Baker, J. W. and Lee, C., (2018). "An Improved Algorithm for Selecting Ground Motions to Match a Conditional Spectrum." *Journal of Earthquake Engineering*, 22(4), 708–723.

[15] Dezhdar, E. and Adebar, P., (2015). "Influence of Ground Motion Scaling on Seismic Response of Concrete Shear Wall Buildings." *The 11th Canadian Conference on Earthquake Engineering*, July 21 – 24, 10, Victoria, BC, Canada.

[16] GEM, (2019). "The OpenQuake-engine User Manual." *Global Earthquake Model (GEM)* OpenQuake Manual for Engine version 3.3.2.

[17] PEER, (2014). "PEER Ground Motion Database." Pacific Earthquake Engineering Research Center, University of California, Berkeley: *https://ngawest2.berkeley.edu*.

[18] S. Mazzoni, F. McKenna, M.H. Scott, G.L. Fenves, (2006). Open system for earthquake engineering simulation (OpenSEES), User Command-Language Manual.

[19] Popovics, S., (1973). "A numerical approach to the complete stress-strain curve of concrete." *Cement and Concrete Research* 3, 583–599.

[20] Son Vu, N., Li, B., and Beyer, K., (2014). "Effective stiffness of reinforced concrete coupling beams." *Engineering Structures* 76, 371–382.

[21] American Society of Civil Engineers, (2014). *ASCE standard, ASCE/SEI 41-13: Seismic evaluation and retrofit of existing buildings*. American Society of Civil Engineers, Reston, Virginia.