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Parametric study on higher mode amplification effects in ductile RC cantilever walls designed for western and eastern Canada

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Abstract: This paper presents a comparison of the results obtained from a parametric study investigating the influence of various parameters on the higher mode amplification effects, and hence on the seismic force demand, in regular ductile reinforced concrete cantilever walls designed with the 2010 National building code of Canada and the 2004 Canadian Standards Association (CSA) standard A23.3 for western and eastern Canada. The studied parameters are the number of storeys, fundamental lateral period, wall aspect ratio, wall cross-section, wall base flexural overstrength, site class and seismic zone. The study is based on inelastic time-history analyses performed with a multilayer beam model and a smeared membrane model based on the Modified Compression Field Theory, and hence accounting for inelastic shear-flexure-axial force interaction. The results are based on more than 9000 inelastic time-history analyses carried out with earthquake ground motions compatible with typical uniform hazard spectra for the western and eastern Canadian regions.

1 Introduction

The seismic force response of reinforced concrete (RC) shear wall structures used as seismic force resisting system (SFRS) for multi-storey buildings is generally dominated by the lateral modes of vibration higher than the fundamental lateral mode on which is traditionally based the common static code procedure for seismic design. The predominating contribution of higher lateral modes in the elastic response of such walls produces moment and shear force demand profiles over the wall height significantly different from and larger (especially at the wall base for shear and at the upper storeys for flexure) than those resulting from the static code procedure. These well-known effects are associated to elastic effects of higher lateral modes. An additional dynamic effect occurs when the wall response changes from elastic to inelastic because the relative contribution of higher lateral modes (primarily that of the second mode) increases while the first-mode contribution saturates and reduces with the first-mode period lengthening (Seneviratna and Krawinkler 1994, Priestley 2003, Sangarayakul and Warnitchai 2004). This dynamic amplification is associated to inelastic effects of higher lateral modes.

Starting with the 2005 edition, dynamic analysis is the default seismic design method in the National Building Code of Canada (NBCC). Dynamic effects can now directly be accounted for while designing building structures. However, the dynamic amplification associated to inelastic effects of higher lateral modes in RC shear walls are usually not taken into account because linear dynamic analysis is the preferred seismic design method by the NBCC (NRCC 2010) which does not recommend any method to estimate this inelastic amplification. Such a method is not even recommended in the 2004 edition of the Canadian Standards Association Standard Design of concrete structures (CSA A23.3-04) (CSA 2004). This standard, however, requires that the inelastic higher mode amplification effects be accounted for when determining capacity design shear forces for ductile walls. Consequently, the capacity design methods prescribed by this standard for seismic design of ductile walls can produce capacity design strength envelopes that fail to conservatively estimate wall shear force demand and to prevent unintended plastic hinge formation at the upper storeys of structurally regular cantilever walls designed to develop a single plastic hinge (SPH) mechanism at the base (Boivin and Paultre 2012a).

With the objective of proposing new capacity design methods for CSA A23.3 that adequately account for higher mode amplifications effects for SPH ductile wall designs, Boivin and Paultre (2012a) identified from a literature review and an extensive parametric study the main parameters and their influence on higher mode effects in isolated ductile cantilever walls. From the studied parameters, the fundamental lateral period and the flexural overstrength at the wall base showed to be the most influencing parameters. However, the study was based on earthquake ground motions representative of the seismic hazard of western Canadian regions where the earthquake ground motions have typically low frequency content. Since seismic zone has a significant influence on seismic response, the results of the study may not necessarily apply to walls designed in eastern Canada where the earthquake ground motions have typically high frequency content. Such difference may imply adapting the capacity design methods proposed by Boivin and Paultre (2012b) in order to suit both western and eastern Canada.

In this regard, this work extends the parametric study carried out by Boivin and Paultre (2012a) to eastern Canada. The methodology of the parametric study is first briefly presented, followed by a comparison of the main results of this study with those obtained by Boivin and Paultre (2012a), and finally a discussion on these results and their implementation in the capacity design methods proposed by Boivin and Paultre (2012b).

2 Methodology

The methodology adopted for the parametric study is that of Boivin and Paultre (2012a) which is as follows: (1) selecting the parameters to be studied and setting them a value range; (2) designing and detailing for seismic forces each studied ductile regular cantilever wall case with the 2010 NBCC and CSA standard A23.3-04 to meet the parameter values associated to that case; and (3) modeling and simulating numerically the inelastic seismic response of each studied case using two different modeling approaches, a simple one and a more realistic one. The previous stages are briefly outlined in the following sections. More details can be found in Boivin and Paultre (2012a).

2.1 Studied parameters

The parameters considered for the study are: number of storeys (N), fundamental lateral period (T), design displacement ductility ratio (μ_Δ), flexural overstrength at the wall base (γ_w), wall aspect ratio (A_r), wall cross-section (WCS), site class (SC) and seismic zone. The values considered for the studied varying parameters are tabulated in Table 1. The μ_Δ ratio and the seismic zone are fixed for the study. In the 2010 NBCC, the μ_Δ ratio corresponds to the product of the ductility-related and overstrength-related force reduction factors R_d and R_o, respectively, using the equal displacement assumption. For the ductile RC cantilever walls studied, the product R_dR_o= 3.5 × 1.6 = 5.6.

Although the city of Montréal (MTL) has the highest urban seismic risk of eastern Canada, the seismicity of the city of Rivière-du-Loup (RDL) was selected as the studied seismic zone for three reasons. First, this city is located in the Charlevoix seismic zone which is the most seismically active region of eastern Canada. Second, as shown in Figure 1, the 2010 NBCC uniform hazard spectra (UHS) (2500 year return period earthquake event) for RDL are larger than those for MTL, for any site class, and are similar in the low-period (or high-frequency) range to those for the West coast city of Vancouver (VAN), the city with the highest urban seismic risk in Canada selected by Boivin and Paultre (2012a) for their parametric study. Finally, the elastic higher mode amplification effects on force demand on fixed-base regular cantilever walls located in MTL and RDL are quite similar. This can be observed from the dimensionless M/(VH) ratio where M and V are the elastic moment and shear force at the wall base obtained with the SRSS (*Square Root of the Sum of the Squares*) method, and H is the wall height from the base. This ratio gives an estimation of the probable relative height, from the wall base, of the resultant seismic horizontal lateral force V along the wall height for a wall with a given fundamental lateral period T. The lower this ratio is from 1, the larger the contribution of the higher lateral modes in the force response is, and hence the larger the elastic higher mode amplification effects are. In general, as illustrated in Figure 1, the M/VH ratio values for MTL and RDL are similar and lower than those for VAN, indicating that elastic higher mode amplification effects are larger for RDL than for VAN.

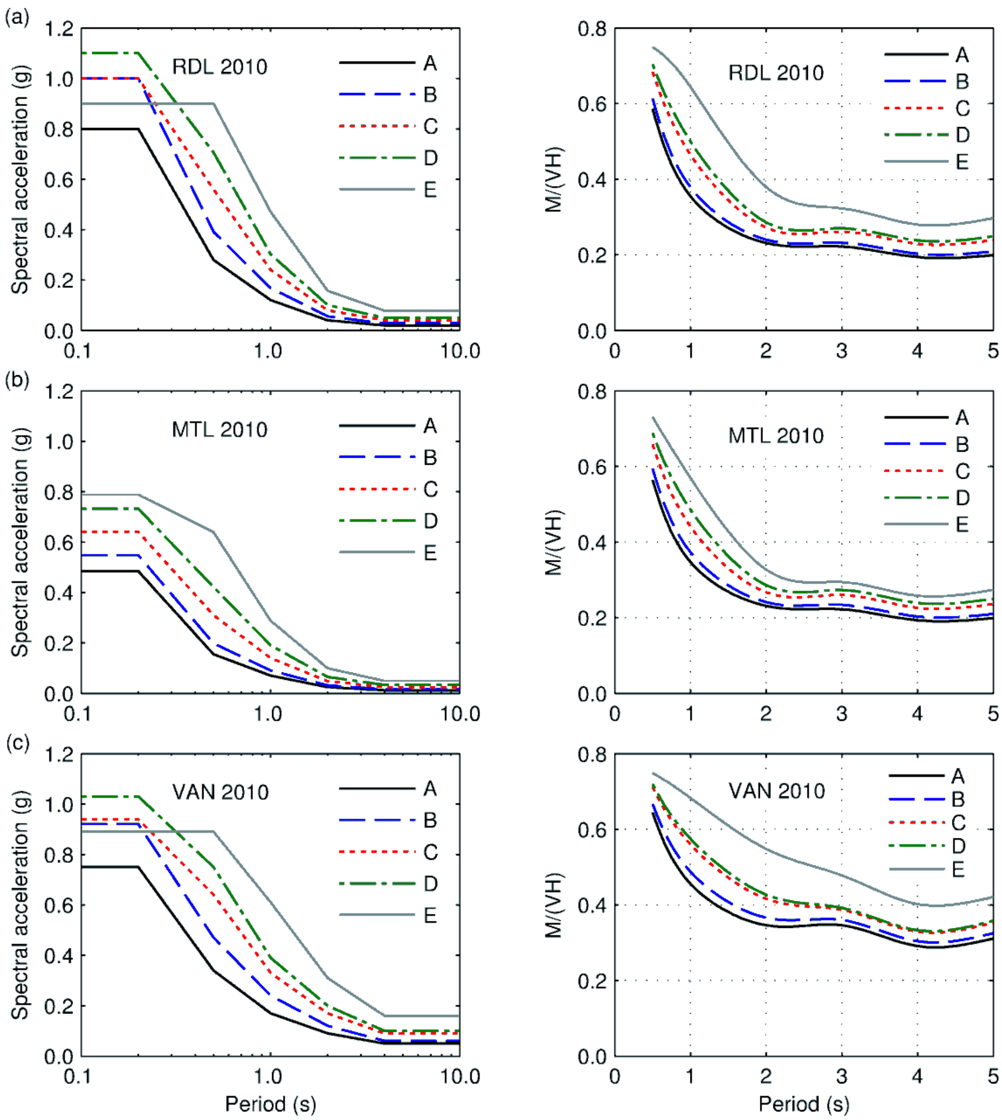

Figure 1 : 2010 NBCC (5%-damped) design spectra and M/VH ratios for (a) Rivière-du-Loup (RDL) (b) Montréal (MTL) and (c) Vancouver (VAN) for different site classes (A: hard rock; B: rock; C: very dense soil and soft rock; D: stiff soil; E: soft soil).

As indicated in Table 1, four of the six site classes defined in the 2010 NBCC were considered for the study. The site class effects were studied only for $N \leq 10$ because soil amplification effects generally reduce with increasing T, assuming a soil largely stiffer than the structure. As illustrated in Figure 2, four different WCSs were considered: rectangular (RT), barbell-shaped (BB) and two I-shaped ones (I1 and H1). For all the studied cases, WCSs were bent about their strong axis.

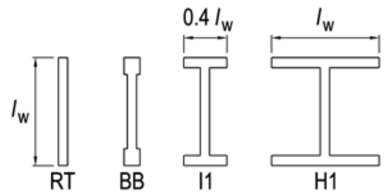

Figure 2: Wall cross-sections (WCSs) studied.

The considered γ_w values range from 1.3 to 4.0, where 1.3 is the minimum value specified by CSA standard A23.3-04 for seismic design of wall structures and 4.0 approximates the theoretical overstrength limit before shear strength design of ductile walls is controlled by the elastic shear forces, which is 5.6/1.3 ≈ 4.3. As the studied walls were designed and detailed such that the reinforcement ratios be as much as possible within the CSA standard A23.3-04 required minimum and maximum reinforcement limits, a γ_w value of 1.3 was not achievable for N values from 20 to 40. This issue finds its explanation mainly in the weak low-frequency content of typical earthquake motions of eastern Canada, unlike those of western Canada. Consequently, only 56 different wall cases, unlike 59 for VAN (Boivin and Paultre 2012a), were studied for RDL, considering that a single SC was used when varying the γ_w values for a given N-T pair and vice versa. Note that the storey height was taken equal to 3.5 m (3.0 m for three cases).

Table 1 : Varying parameter values for the parametric study.

N	T	WCS	A_r	γ_w	SC
5	0.5	RT	3.5	1.3, 2.0, 3.0, 4.0	A, C, D, E
	1.0	RT	5.833	1.3, 2.0, 3.0, 4.0	A, C, D, E
10	1.0	RT	3.5	2.0	A, C, D, E
		RT	4.375	2.0	C
		RT	5.833	1.3, 2.0, 3.0, 4.0	C
	1.5	RT	4.375	2.0	C
		RT	5.833	1.3, 2.0, 3.0, 4.0	A, C, D, E
		RT	8.75	2.0	C
15	1.0	BB	4.375	1.3, 2.0, 3.0, 4.0	D
	2.0	BB	7.0	1.3, 2.0, 3.0, 4.0	D
20	1.5	I1	7.0	2.0	D
	2.0	RT	7.0	2.0	D
		BB	7.0	1.3*, 2.0, 3.0, 4.0	D
		I1	7.0	2.0	D
		H1	7.0	2.0	D
		I1	10.0	2.0	D
30	2.0	I1	8.57	2.0	D
	3.0	I1	10.0	1.3*, 2.0, 3.0, 4.0	D
40	3.0	I1	10.0	2.0	D
	4.0	I1	11.67	1.3*, 2.0, 3.0, 4.0	D

* : only for Vancouver (Boivin and Paultre 2012a)

2.2 Seismic design and detailing

Each studied wall case was designed with the 2010 NBCC and CSA standard A23.3-04 assuming that the isolated regular cantilever wall is fully fixed at ground level and hence that a SPH will develop at the wall base. Seismic design forces were obtained from the modal response spectrum method prescribed by the 2010 NBCC, with the exception that the design base shear force, V_d, was always that resulting from the modal superposition and the force reduction with R_dR_o (NBCC building importance factor, I_E=1). Concrete cracking was accounted for by using the effective section properties recommended by CSA standard A23.3-04. Capacity design was performed according to the CSA standard A23.3-04 to constrain the plastic hinge at the wall base and prevent shear failure. The specified material properties used for design are f'_c = 30 MPa for concrete compressive strength and f_y = 400 MPa for steel yield strength. Typical wall thicknesses varying between 400 mm and 700 mm were used.

2.3 Structural modeling

The simulation of the inelastic seismic response of each studied wall case was performed with a two-dimensional (2D) inelastic time-history analysis (ITHA) using the constant acceleration Newmark method for time integration. Two 2D modeling approaches were adopted to simulate the inelastic response of the studied walls. As shown in Figure 3, two wall models were considered: a force-based multilayer beam-column model where shear deformation is linear elastic and uncoupled from flexure and axial deformations and a smeared membrane model based on the Modified Compression Field Theory, and hence accounting for inelastic shear-flexure-axial force interaction. The analyses with the beam and membrane wall models were conducted with the open-source software framework OpenSees (OS) (version 2.1.0) (Mazzoni et al. 2006) and the finite element analysis program VecTor2 (VT2) (Wong and Vecchio 2002), respectively. P-delta, concrete tension stiffening, concrete strength decay and steel Bauschinger effects were taken into account.

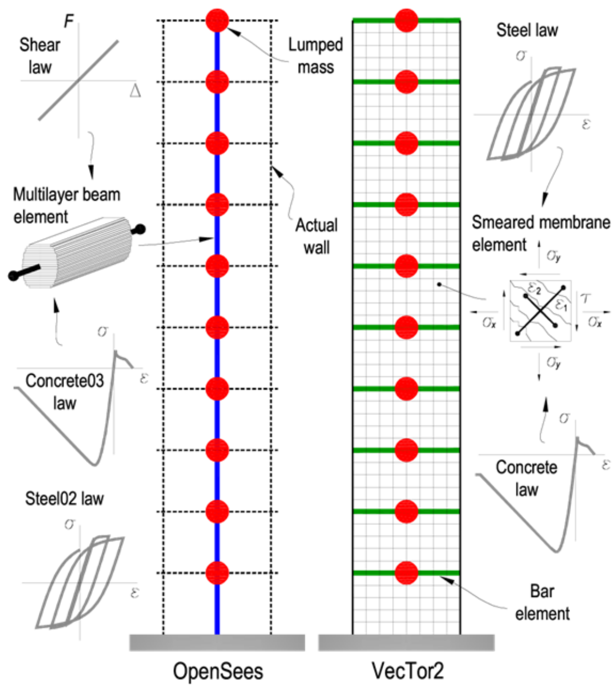

Figure 3 : Structural wall models for ITHA with OpenSees and VecTor2.

The damping model used for ITHA is the initial stiffness-based Rayleigh damping with damping ratios specified at the first and last lateral mode, ensuring that the highest modes of the structure remain sub-critically damped throughout the response. A modal ratio of 2% of critical damping was assigned.

2.4 Input ground motions

A suite of 40 ground motion records per site class was selected from the synthetic earthquake time histories generated by Atkinson (2009) for the 2005 NBCC UHS for eastern and western Canada regions and for site classes A, C, D, and E. Half of the selected records of a suite corresponds to M6 events in the 10-30 km range while the other half corresponds to M7 events in the 15-100 km range. For a given site class, the selected records were matched to the 2010 NBCC UHS for RDL as recommended by Atkinson and over period ranges specific to magnitude-distance scenarios. The resulting UHS-compatible records were used independently in analysis as input horizontal motions only.

3 Inelastic seismic analysis results

3.1 Response parameter definitions

In this section, the seismic responses of the studied wall cases obtained for the eastern Canada (RDL) are presented and compared to those obtained by Boivin and Paultre (2012a) for the western Canada site (VAN). All predicted demands for a given wall case presented in this section are the means obtained from 40 ground motion records. The dynamic shear amplification at a given storey is calculated as the ratio of the mean predicted storey shear force demand to the capacity design (probable) shear force V_p at that storey, with:

[1] $$V_p = V_f \left(\frac{M_p}{M_f} \right)_{base}$$

where V_f is the design shear force, M_f is the design moment at the wall base, both determined from the modal response spectrum method, and M_p is the probable moment resistance at the wall base calculated with f'_c and an equivalent steel yield stress of 1.25 f_y. The dynamic shear amplification is calculated at the wall base and as the average value over all storeys (AOS). In addition to this response parameter, the predicted moment, storey shear force and curvature ductility (μ_φ) demands over the wall height are presented. The moment and storey shear force demands are normalized relative to the nominal base moment resistance ($M_{n\ base}$) and the predicted base shear force demand (V_{base}), respectively. For clarity purposes, the presented curves of curvature ductility demand are envelopes of the maximum curvature ductility of each storey (which always occurs at the storey base for the studied walls), and hence are smoothed mean distributions of the curvature ductility demand along the wall height. Note that only the force responses predicted with VT2 are presented since the OS wall model tends to overestimate forces and produce incorrect shear tendencies (Boivin and Paultre 2012a). Also note that curvature ductility is predicted only with OS by estimating the global yield curvature of a wall section from a tri-linear idealization of its monotonic moment–curvature response. VT2 does not output curvature which is not simple to determine because plane sections do not remain plane due to inelastic shear deformation. Because of the selected moment-curvature idealization, the complete formation of a plastic hinge occurs when $\mu_\varphi \approx 2$.

3.2 Influence of the studied parameters

For either site, RDL or VAN, the flexural overstrength at the wall base (γ_w) and the fundamental lateral period (T) are the studied parameters having the most significant influence on the dynamic shear amplification and the seismic force demand on the studied wall cases. The influence of these parameters on force demand is illustrated in Figure 5 and Figure 6 for RDL and VAN, respectively. These figures show that, regardless of N and T values, dynamic shear amplification rapidly decreases, almost linearly sometimes, with increasing γ_w. For both RDL and VAN, Figure 4 illustrates, for a given wall overstrength factor value, the maximum dynamic shear amplification predictions presented in Figure 5 and Figure 6 for each considered T value. Figure 4 shows similar values for RDL and VAN with the difference that, for VAN, dynamic shear amplification largely increases with increasing T for T values between 0.5 s and 1.0 s whereas, for RDL, this increase is almost negligible, except for γ_w = 1.3, and hence the amplification values for T = 0.5 s for RDL are greater than those for VAN, which are about 1.0. For T > 1.0 s, this figure indicates in general that, regardless of the site, dynamic shear amplification slightly increases with T for $\gamma_w \leq 2.0$ but remains almost constant for $\gamma_w \geq 3.0$. Note that the reductions of base shear amplification observed in Figure 4 for γ_w = 1.3 and T > 1.0 s result of significant flexural yielding ($\mu_\varphi > 2$) of the walls at the upper storeys, as illustrated in Figure 7. Although γ_w affects dynamic shear amplification, both Figure 5 and Figure 6 show that γ_w has no significant influence on the storey shear force profile along the wall height for T ≥ 1.0, resulting in very similar profiles.

For the flexural demand, Figure 5 and Figure 6 show that as T increases so does the flexural demand, particularly at the upper storeys, but as γ_w increases, this demand reduces rapidly without, however, inhibiting the plastic hinge mechanism at the wall base, even for γ_w = 4.0, as illustrated in Figure 7 for both sites. Figure 5 and Figure 6 indicate that the flexural demand at upper storeys is significantly larger for RDL than for VAN as the higher mode contribution to flexural response is larger for RDL, which is typical for eastern Canada regions. Figure 7 shows that in general no plastic hinge mechanism is predicted at the upper storeys ($\mu_\varphi < 2$) for $\gamma_w \geq 2.0$ despite sometimes light flexural yielding. Figure 5d and Figure 6d show a certain match between the moment demand profiles for T=2.0 s and 4.0 s and that the moment profiles associated to T=4.0 s become slightly lower than those associated to T=2.0 s as γ_w increases. Similar observations can be made for curvature ductility demand (see Figure 7d). This suggests that the higher mode contribution to flexural response saturates for T>2.0 s and its relative influence on response becomes less as γ_w increases.

From Figure 7, note that, for $\gamma_w \geq 2.0$, the plasticity height at the wall base decreases, with respect to wall height (H), from about 20% to 2.5% of H as N (or H) and γ_w increase. This differs from the relation $0.5l_w + 0.1H$ (where l_w is the horizontal wall length) prescribed by CSA standard A23.3-04 for determining the base plastic hinge height requiring special ductile detailing, where the estimated plastic hinge height is at least 10% of H and independent of γ_w. This shows that this relation is inadequate, giving too conservative base plastic hinge height estimates for tall walls with large flexural overstrength at the wall base. In general Figure 7 indicates that the predicted plastic hinge heights for VAN are higher than those for RDL mainly because the relative contribution of the first lateral mode to flexural response is more significant for VAN than for RDL.

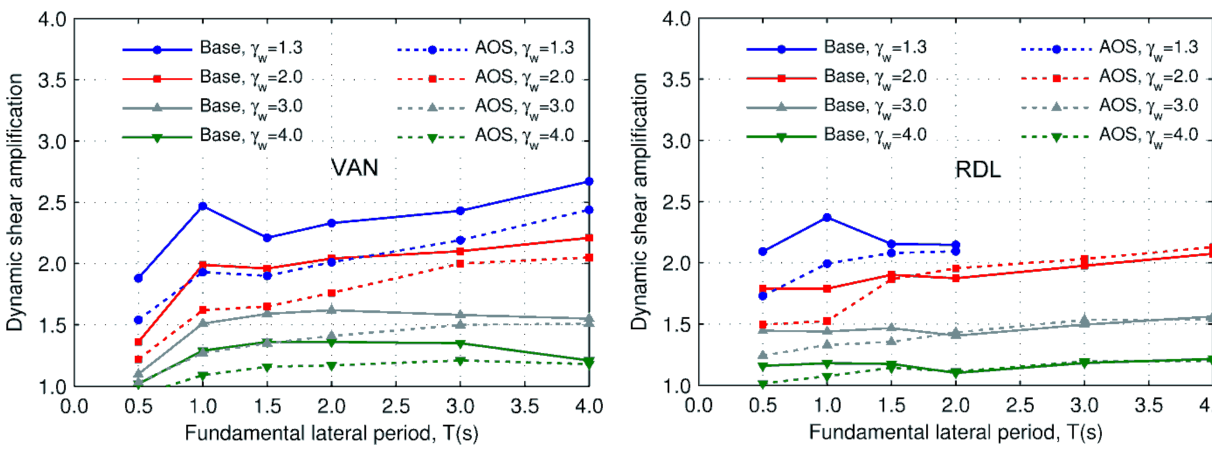

Figure 4 : Predicted maximum dynamic shear amplifications at the wall base and over all storeys (AOS) for wall overstrength factor (γ_w) values equal to 1.3, 2.0, 3.0 and 4.0 for Vancouver (VAN) and Rivière-du-Loup (RDL).

4 Discussion

4.1 Flexural strength design

The results obtained in this work show that T and γ_w influence the flexural moment demand, which increases with increasing T and reducing γ_w, for both considered sites. Also they show significant flexural yielding at the upper storeys of the studied walls for T ≥ 1.0 s and γ_w = 1.3, regardless of the site. This means that the predicted flexural demand at these locations has exceeded the provided flexural strength, and hence the capacity design moment envelope. Actually, for T ≥ 1.0 s and γ_w < 3.0, the predicted flexural demand at the upper storeys has always significantly exceeded the capacity design moment envelope, though this excess reduces with increasing γ_w. This shows once more that the capacity design method prescribed by CSA standard A23.3-04 for flexural strength design can produce inadequate design envelopes. Nevertheless, the obtained results show that, for γ_w ≥ 2.0 and any T, the plastic hinge mechanism is generally constrained at the wall base, as expected, despite sometimes light flexural yielding at the upper storeys. All these results suggest that a SPH design may be inadequate for regular ductile cantilever walls with γ_w < 2.0. It is for this reason that the new method proposed by Boivin and Paultre (2012b) for determining a capacity design moment envelope for flexural strength design of such walls requires that γ_w ≥ 2.0 instead of 1.3, as required by CSA standard A23.3-04.

Despite the similar flexural results obtained for RDL and VAN, the predicted moment profiles along the wall height for RDL differ, particularly at the upper storeys, from those for VAN, for a given T and γ_w. This difference implies modification of values of some parameters defining the capacity design moment envelope proposed by Boivin and Paultre (2012b) because the current proposed values are for walls designed for western Canada. As illustrated in Figure 8a, the proposed design envelope is a simple bilinear envelope that requires determining only two parameters once the required flexural reinforcement at the wall base has been set: the moment ratio α_M of mid-height overstrength moment, $M_{0.5H}$, to the nominal moment capacity at the wall base, M_{nb}, and the plastic hinge height h_p over which special ductile detailing is required. From the maximum moment ratios at wall mid-height given in Figure 5, design α_M values are proposed for eastern Canada and presented in Figure 9b. The comparison of these values with those proposed for western Canada (see Figure 9a) shows that the latter values are significantly lower than those for eastern Canada, except when the wall base overstrength is large (R_dR_o/γ_w=1.4).

Boivin and Paultre (2012b) proposed the following relation for estimating h_p:

$$[2] \quad h_p = 0.8h_s + \beta_0 H \geq \max\left(h_s; 0.5l_w\right)$$

where h_s is the storey height, l_w and H are the wall length and height, respectively, and β_o is equal, for western Canada, to 0.10, 0.05 and 0.03 for R_dR_o/γ_w values of 2.8, 1.87 and 1.4, respectively. The corresponding β_o values derived from the results obtained in this work for RDL are 0.07, 0.015, and 0.01. Since these values are lower than and not so different from those for western sites, the β_o values proposed for western Canada are also conservatively proposed for eastern Canada.

4.2 Shear strength design

The shear results obtained in this work show that, regardless of the site, γ_w has no significant influence on the shear force demand profile for T ≥ 1.0 s while, for any T, dynamic shear amplification largely reduces with increasing γ_w. The predicted dynamic shear amplification values significantly larger than unity for γ_w < 4.0 (see Figure 4) shows that the capacity design method prescribed by CSA standard A23.3-04 for shear strength design of ductile walls can produce inadequate design envelopes for both western and eastern Canadian sites. The different shear amplification values between both sites imply that the new method proposed by Boivin and Paultre (2012b) for determining a capacity design shear envelope for shear strength design of regular ductile cantilever walls be adapted for eastern Canada. The proposed design envelope is illustrated in Figure 8b, with the capacity design base shear force, V_{pb}, given by:

$$[3] \quad V_{pb} = \bar{\omega}_v V_{p\,base} \leq V_{limit\,base}$$

where $\bar{\omega}_v$ is a dynamic shear amplification factor, $V_{p\,base}$ is the probable shear force V_p (Eq. [1]) at the wall base, $V_{limit\,base}$ is the base shear force limit determined from the elastic shear forces and reduced with R_dR_o = 1.3, as specified by CSA standard A23.3-04; and with a height ratio $\bar{\xi}$ given by $0.5 \leq \bar{\xi} = 1.5 - T \leq 1.0$. The results obtained in this work for RDL indicate that the latter relation for $\bar{\xi}$ is adequate also for eastern Canada. It is, however, not possible to extend the proposed $\bar{\omega}_v$ values for western Canada (see Figure 10a) to eastern Canada. As shown in Figure 4, a $\bar{\omega}_v$ value of 1.0 is reasonable for T = 0.5 s and γ_w ≥ 2.0 for VAN but is not for RDL, except for γ_w = 4.0. In order to identify the trend of dynamic shear amplification for T < 0.5 s for RDL, new analyses were carried out for walls with T = 0.2, 0.3 and 0.4 s. Although not shown, the results of these analyses show that dynamic shear amplification is either inexistent or negligible for T = 0.2 s and linearly increases with increasing T. Note that varying the damping values from 2% to 4% during the analyses did not change significantly the previous results. The combination of these results with those given in Figure 4 for RDL enables to propose $\bar{\omega}_v$ values for eastern Canada, which are presented in Figure 10b.

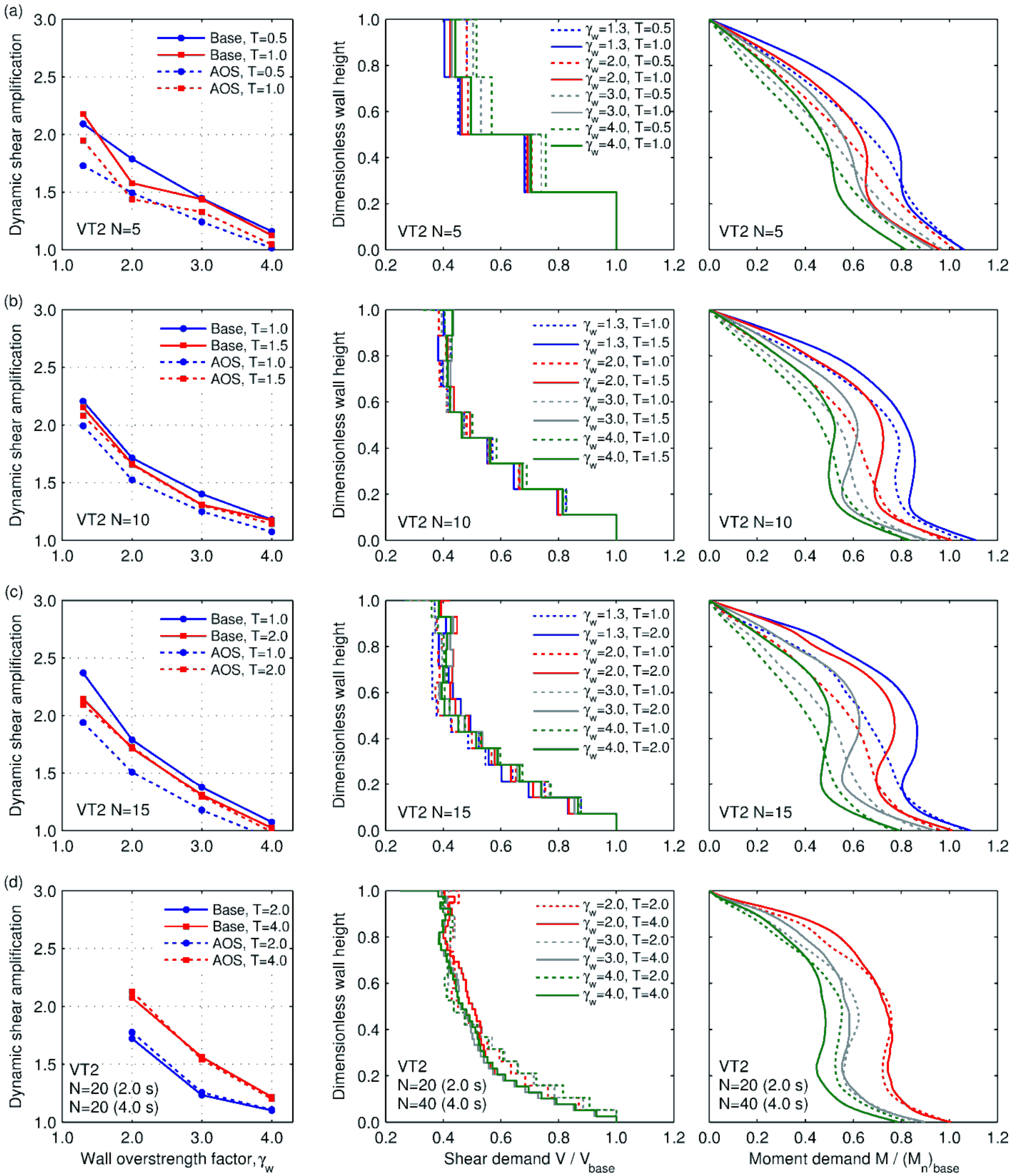

Figure 5 : Influence of wall base overstrength (γ_w) on dynamic shear amplification and shear force and moment demands for Rivière-du-Loup (RDL): (a) N=5; (b) N=10; (c) N=15; (d) N=20 and 40.

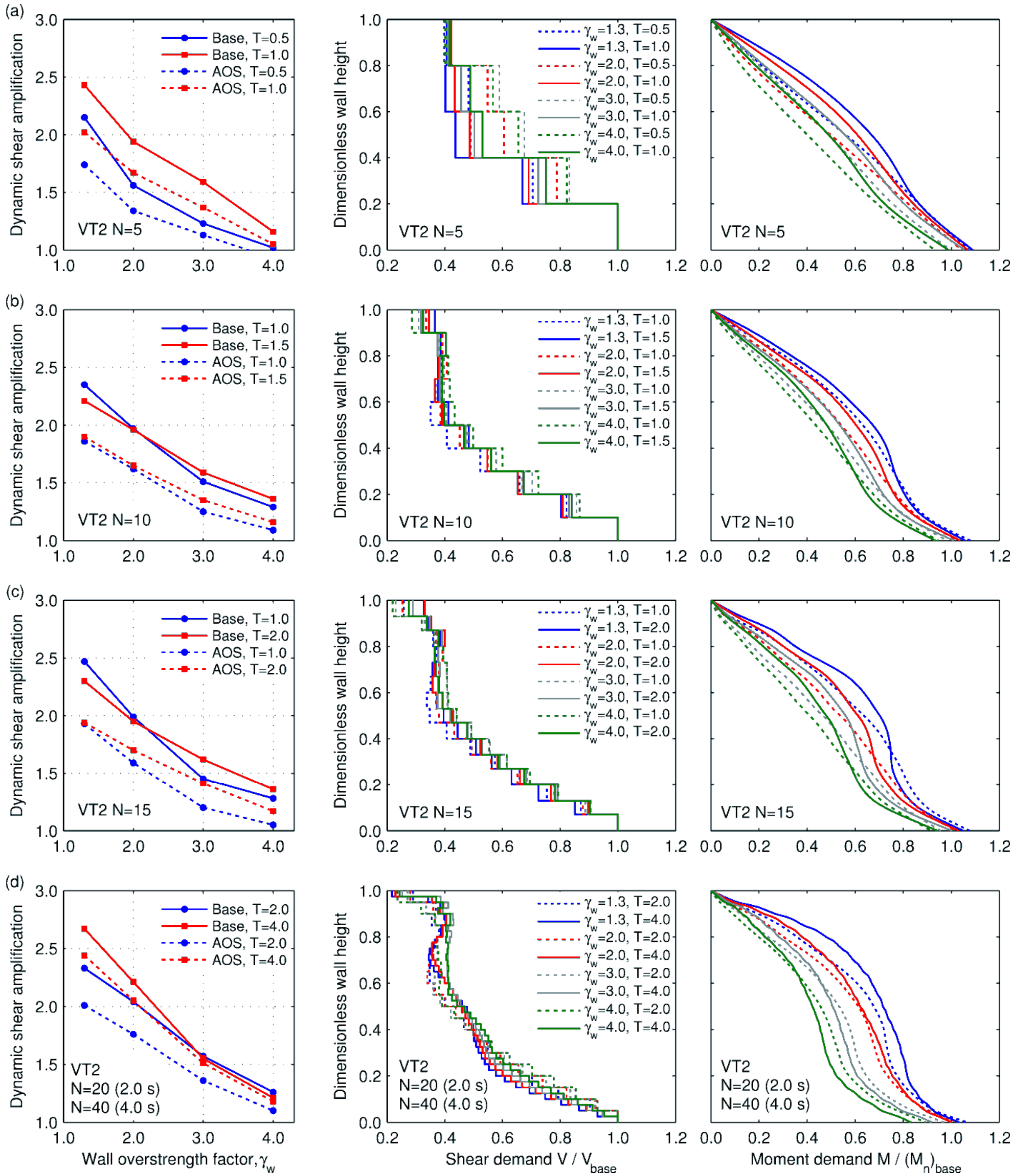

Figure 6 : Influence of wall base overstrength (γ_w) on dynamic shear amplification and shear force and moment demands for Vancouver (VAN): (a) N=5; (b) N=10; (c) N=15; (d) N=20 and 40.

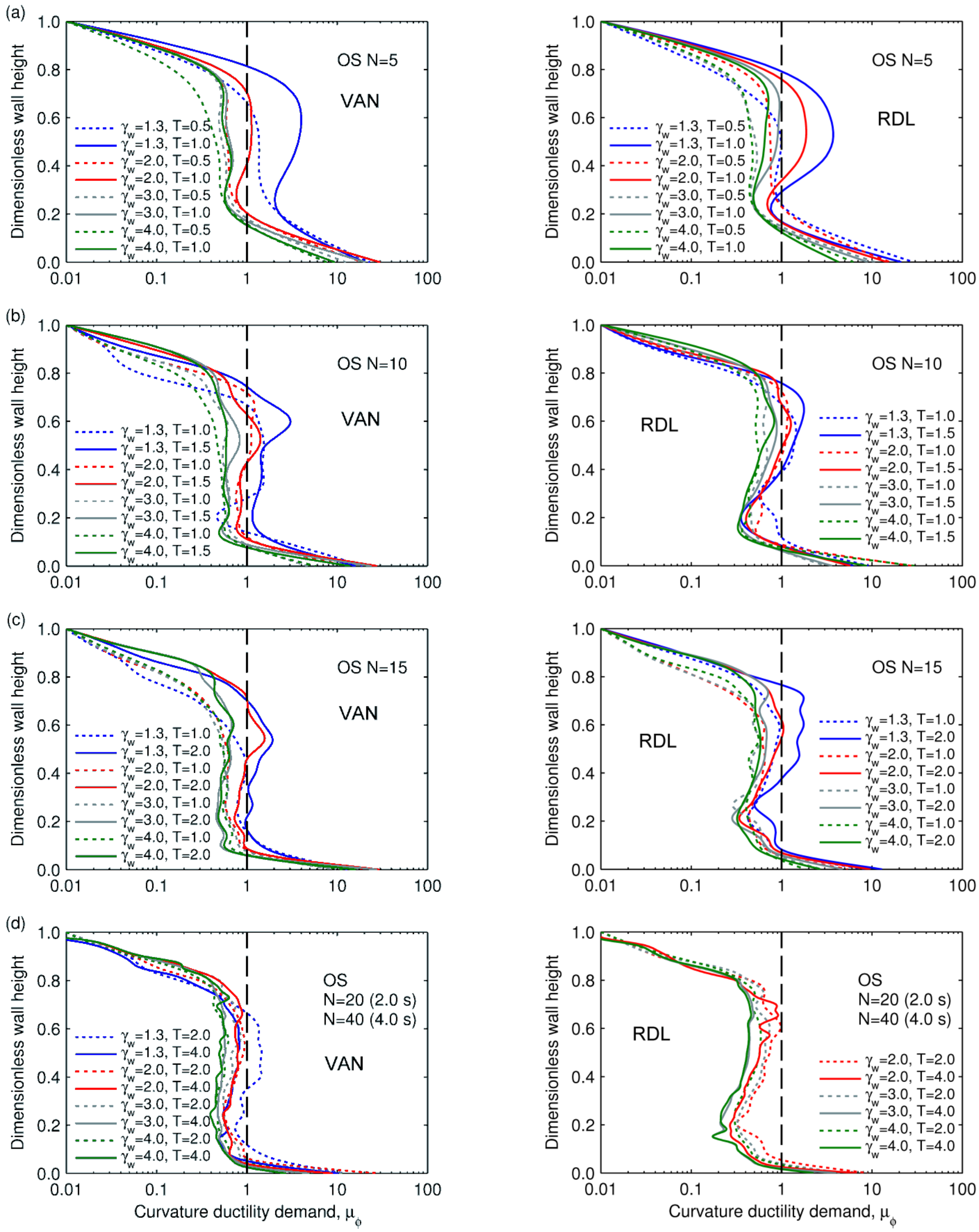

Figure 7 : Influence of wall base overstrength (γ_w) on curvature ductility demand (μ_φ = 1 ≡ sectional yielding) for Vancouver (VAN) and Rivière-du-Loup (RDL): (a) N=5; (b) N=10; (c) N=15; (d) N=20 and 40.

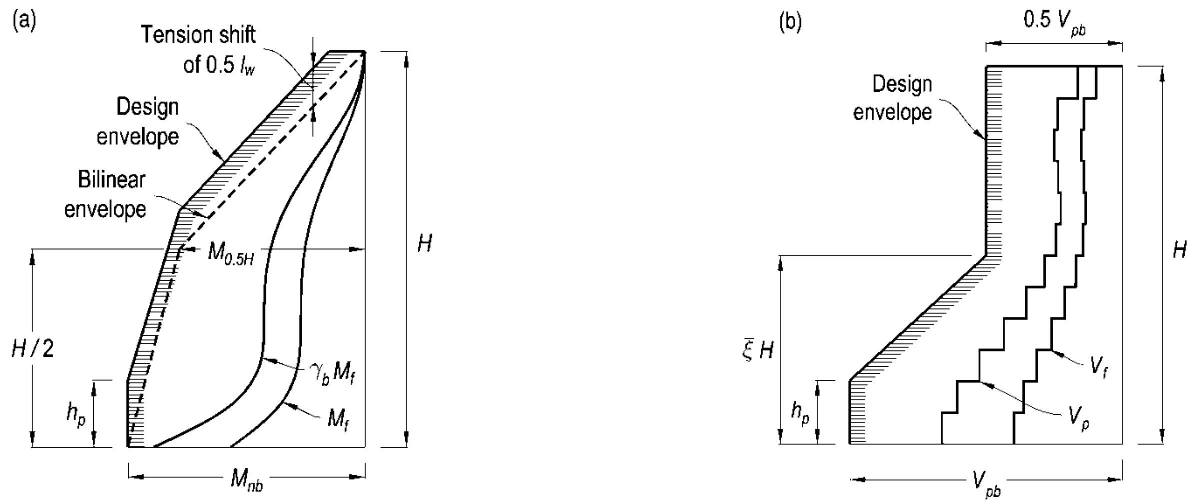

Figure 8 : Proposed capacity design envelopes: (a) flexural strength design; (b) shear strength design.

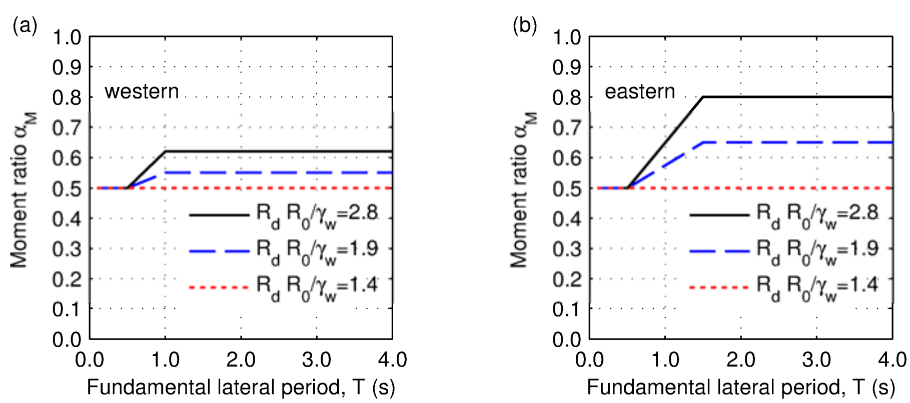

Figure 9 : Proposed α_M values : (a) western Canada (b) eastern Canada

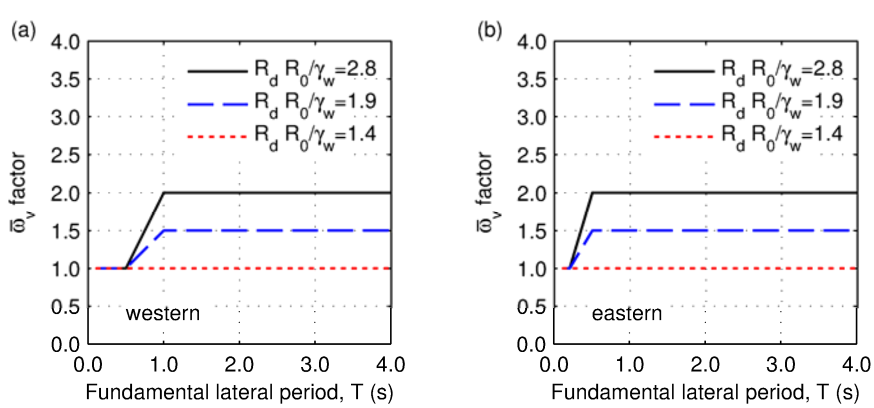

Figure 10 : Proposed $\bar{\omega}_v$ values : (a) western Canada (b) eastern Canada

5 Conclusion

This work extends the parametric study carried out by Boivin and Paultre (2012a) to eastern Canada. It is based on ITHAs carried out with a large suite of design-level ground motions and several fixed-base isolated walls modelled with two different 2D modeling approaches: a multilayer beam approach (OpenSees), modeling shear deformation linearly and uncoupled to flexure and axial deformations, and a smeared membrane element approach (VecTor2), modeling shear deformation inelastically and fully coupled with the flexure–axial interaction. The seismicity of the city of Rivière-du-Loup (RDL) was selected for the study because its seismic hazard is representative of that of eastern Canadian cities. Comparisons with the results obtained for Vancouver (VAN), the West coast city with the highest urban seismic risk in Canada, were made. The difference of seismicity between RDL and VAN showed, as expected, a significantly larger higher mode contribution to wall force response for RDL than for VAN. Among the studied parameters, the seismic zone, the flexural overstrength at the wall base (γ_w) and the fundamental lateral period (T) of the wall are those having the most significant influence on the higher mode amplification effects, and hence on the wall force demand. The capacity design methods proposed by Boivin and Paultre (2012b) now accounts for the difference of seismicity between western and eastern Canada.

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