Seismic force demand on ductile reinforced concrete shear walls subjected to western North American ground motions: Part 2 — new capacity design methods

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Abstract: This paper proposes for the Canadian Standards Association (CSA) standard A23.3 new capacity design methods, accounting for higher mode amplification effects, for determining, for a single plastic hinge design, capacity design envelopes for flexural and shear strength design of regular ductile reinforced concrete cantilever walls used as seismic force resisting system for multistorey buildings. The derivation of these methods is based on the outcomes from a review on various capacity design methods proposed in the current literature and recommended by design codes and from the extensive parametric study presented in the companion paper. A discussion on the limitations of the proposed methods and on their applicability to various wall systems is presented.

Key words: CSA standard A23.3, capacity design method, ductile concrete cantilever wall, seismic force demand, higher mode amplification effects.

Résumé : Cet article propose pour la norme CSA A23,3 de nouvelles méthodes de dimensionnement à la capacité considérant les effets d’amplification des modes supérieurs pour déterminer des enveloppes de dimensionnement en flexion et en cisaillement pour des murs ductiles réguliers en béton armé à rotule plastique unique à la base utilisés comme système de résistance aux forces sismiques de bâtiments multi-étages. La dérivation de ces méthodes est basée sur les résultats d’une revue de diverses méthodes de dimensionnement à la capacité proposées dans la littérature et recommandées par des codes de conception, et d’une étude paramétrique présentée dans l’article complémentaire. Une discussion sur les limitations des méthodes proposées et sur leur applicabilité pour divers systèmes de murs est présentée.

Mots-clés : norme CSA A23,3, méthode de dimensionnement à la capacité, mur ductile en béton armé, demande sismique en force, effets d'amplification des modes supérieurs.

1. Introduction

To produce economical seismic designs, the modern building codes allow reducing seismic design forces if the seismic force resisting system (SFRS) of a building is designed to develop an identified mechanism of inelastic lateral response. To ensure that the inelastic mechanism develops as intended and no undesirable failure modes occur, the identified inelastic zones of the SFRS, commonly named plastic hinges, are specially designed and detailed to possess sufficient flexural ductility and all other regions of the structure and other possible behavior modes are provided with sufficient strength. This design approach, referred to as capacity design, is implemented in the Canadian Standards Association (CSA) standard A23.3 since the 1984 edition for seismic design of ductile reinforced concrete (RC) shear walls with the objectives of providing sufficient flexural strength to confine the inelastic mechanism to identified flexural plastic hinges and sufficient shear strength to ensure a flexure-governed inelastic lateral response of the walls. To fulfill these objectives, CSA standard A23.3 and its Commentary specify, for regular wall structures, capacity design methods for determining capacity design shear and moment envelopes over the height of the wall assuming the development of a single plastic hinge at the wall base. This design is referred to as single plastic hinge (SPH) design.

Boivin and Paultre (2012), in the companion paper (referred to herein as only the companion paper), showed from an extensive parametric study that the capacity design methods prescribed by the 2004 edition of the CSA standard A23.3 (A23.3-04) (CSA 2004) for the SPH design of ductile RC walls can produce capacity design envelopes that fail, for design-level seismic motions, to conservatively estimate the wall shear force demand and prevent unintended plastic hinge formation at the upper storeys of regular multistorey ductile cantilever walls whose seismic force response is dominated by lateral modes of vibration higher than the fundamental lateral mode and whose level of flexural overstrength at the wall base is low. These underestimation issues result from deficient capacity design considerations regarding higher mode amplification effects in such walls.

In this regard, this paper proposes for CSA standard A23.3...
new capacity design methods, accounting for higher mode amplification effects, for determining, for a SPH design, adequate capacity design envelopes for flexural and shear strength design of regular ductile RC cantilever walls used as SFRS for multistorey buildings. This paper presents first a short review of various capacity design methods proposed in the current literature and recommended by design codes for determining capacity design moment and shear envelopes, followed by the presentation of the new methods proposed for CSA standard A23.3 and finally a discussion on the limitations of these new methods and their applicability to various wall systems. The derivation of the new methods is based on the outcomes from the literature review and the parametric study presented in the companion paper.

Note that other design concepts, such as the dual plastic hinge (DPH) design concept (Panagiotou and Restrepo 2009; Ghorbanirenani 2010), have been proposed to account for higher mode amplification effects in seismic design of ductile walls. The SPH design concept, however, was preferred for the proposed capacity design methods because CSA standard A23.3 is based on this design concept and therefore, the proposed changes to current seismic provisions are relatively minor. A DPH design would imply for instance the derivation of new force reduction factor values and new ductility check methods for ductile walls.

2. Review of capacity design methods

In this section, various capacity design methods proposed in the current literature and recommended by design codes for determining capacity design moment and shear envelopes for a SPH design are outlined as well as their limitations in estimating the seismic force demand on ductile walls whose seismic force response is governed by higher mode responses. Most of these methods are for a conventional force-based design (FBD) while the others are for a displacement-based design (DBD). All reviewed methods were primarily developed for RC cantilever wall structures regular and uniform in strength and stiffness over the height of the building by assuming the development of a SPH mechanism at the wall base. In addition, the methods were developed considering that the seismic design forces are determined from linear elastic analysis. Furthermore, they generally assume that the designed cantilever wall, isolated or part of a wall system, is the sole SFRS of the building in the direction under consideration. Finally their application requires that the design wall bending moment and shear force diagrams over the wall height, \( M_f \) and \( V_f \), respectively, be pre-determined from a seismic FBD or DBD procedure, static or dynamic, by taking into account, when applicable, any factors required by the code and force redistribution between the walls. For comparison purposes, the capacity design methods prescribed by CSA standard A23.3-04 are outlined and possible capacity design envelopes resulting from their application are illustrated. It is noted that the latest edition (2008) of the American Concrete Institute (ACI) standard 318 for structural concrete still does not specify any capacity design method for seismic design of ductile RC walls.

Note that, since the 2005 edition, the National Building Code of Canada (NBCC) (NRCC 2010) prescribes a force reduction factor \( R_dR_o \) when determining seismic design forces (\( M_d, V_d \)) from linear elastic analysis, where \( R_d \) and \( R_o \) are the ductility-related and overstrength-related force reduction factors, respectively. For ductile RC cantilever walls, \( R_d = 3.5 \) and \( R_o = 1.6 \), which gives \( R_dR_o = 5.6 \).

2.1. Flexural strength design

The capacity design methods presented in this section aim to prevent the formation of unintended plastic hinges above the expected plastic hinge zone at the wall base. Prior to applying these methods, the critical section at the wall base has to be designed and detailed such that the moment resistance at this section is at least equal to \( M_f \). Note that the methods are generally based on one of the following flexural resistances: factored, nominal or probable. The nominal moment resistance, \( M_{\text{fn}} \), is calculated with either specified values, as required by CSA standard A23.3-04, or characteristic values for material strengths while the factored moment resistance, \( M_f \), is calculated with material strengths reduced by partial safety factors lower than unity. The probable moment resistance, \( M_p \), as defined in CSA standard A23.3-04, is calculated with material resistance factors equal to unity and an equivalent steel yield stress of 1.25 times its specified value to account for development of strain hardening in tensile reinforcing steel. It is important to add that the flexural overstrength of a wall in CSA standard A23.3-04 is estimated with the wall overstrength factor \( \gamma_w \) taken as the ratio of \( M_p/M_f \) at the wall base and not less than 1.3.

2.1.1. CSA standard A23.3-04

Since the 2004 edition, CSA standard A23.3 prescribes for ductile walls a method for determining a capacity design moment envelope. This method consists in amplifying \( M_f \) above the assumed plastic hinge region \( h_p \), calculated as \( 0.5l_w + 0.1H \), by the ratio \( M_p/M_f \) calculated at the top of \( h_p \), where \( l_w \) and \( H \) are the wall length and height, respectively (see Fig. 1a). The resulting design envelope has essentially the same profile above \( h_p \) as that of \( M_f \). Despite capacity design requirements, the 1994 edition of CSA standard A23.3 (A23.3-94) did not specify any capacity design method for determining design envelopes. However, the Explanatory notes on CSA standard A23.3-94 (CAC 1995) recommended a probable moment envelope varying linearly from the top of \( h_p \) to the top of the wall, as illustrated in Fig. 1a. Various works (Tremblay et al. 2001; Boivin 2006), however, showed that the linear probable envelope is inadequate to prevent the formation of unintended plastic hinges at the upper storeys of regular cantilever walls whose flexural response is governed by the higher mode responses.

2.1.2. Paulay and Priestley (1992)

The capacity design moment envelope recommended by Paulay and Priestley (1992) is determined by assuming a moment envelope varying linearly from the nominal moment strength at the base to zero strength at the top of the wall, and by vertically translating this linear envelope by a distance equal to \( l_w \) to account for tension shift effects resulting from inclined flexure-shear cracking (diagonal tension), as shown in Fig. 1b. A minimum nominal strength, calculated with the required minimum reinforcement and zero axial load, is to be considered at the top of the wall. Based on Paulay and Priestley, the linear envelope is assumed to take into account the...
contribution of higher modes in the bending moments over the entire height of the wall. As shown by Bachmann and Linde (1995), this design envelope, however, is inadequate for walls whose flexural response is governed by higher mode responses because it cannot capture the flexural demand increase above the base hinging region produced by the higher lateral modes.

2.1.3. 2004 Eurocode 8

As a simplified procedure, the 2004 edition of the Eurocode 8 (EC8) (CEN 2004) specifies that the design moment envelope along the height of the wall should be given by an envelope of \( M_f \), vertically displaced to account for tension shift, as shown in Fig. 1. A linear envelope can be used if the structure does not exhibit significant discontinuities of mass, stiffness or resistance over its height. In such case, the resulting design envelope would be similar to that recommended by Paulay and Priestley (1992). Although required, EC8 does not specifically provide any method or relation to estimate tension shift. This shift can be approximated with the height of the critical region, \( h_{cr} \), which may be estimated as \( h_{cr} = \max(l_w; H/6) \) but need not be greater than \( 2l_{w} \) or \( h_s \), for structures with less than 7 storeys, and 2\( h_s \), for structures with 7 storeys or more, where \( h_s \) is the clear storey height. It is important to note that, unlike the other reviewed methods, the 2004 EC8 method makes use of the base design bending moment rather than the base bending strength to generate the design envelope. Consequently, this method is inadequate for capturing the relative higher mode contribution in the flexural demand at the upper storeys because this contribution depends on the base flexural overstrength, as shown in the companion paper.


The capacity design moment envelope proposed by Bachmann and Linde (1995) aims to overcome the limitation of the design envelope recommended by Paulay and Priestley (1992) with regard to higher mode amplification effects. As shown in Fig. 1d, their envelope presents a constant strength \( R_p \) over an assumed base hinging region of height \( l_{w} \), which can be estimated as the larger of \( l_{w} \) or \( H/6 \). \( R_p \) is set equal to or greater than \( g_R M_f \), where \( g_R \) is the resistance factor taken as 1.2. As seen from Fig. 1d, an increased strength is suggested immediately above the hinging region to prevent yielding in the upper part of the wall. For this region extending over a height \( l_{ec} \), the required strength \( R_e \) is kept constant and is equal to \( \lambda_f R_p \), where \( \lambda_f \) is the flexural overstrength factor usually taken as 1.2. The height \( l_{ec} \) depends on how slender the wall is. It is taken as a fraction \( \alpha_{ec} \) of the total height of the elastic region \( l_e \) according to \( l_{ec} = \alpha_{ec} l_e = 0.20T_1 l_e \), where \( T_1 \) is the fundamental lateral period of the wall in the direction under consideration. Above the height \( l_{ec} \), the linear

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Fig. 1. Capacity design moment envelopes: (a) CSA standard A23.3-04 and 1995 CAC; (b) Paulay and Priestley (1992); (c) 2004 EC8; (d) Bachmann and Linde (1995); (e) Priestley and Amaris (2003); (f) Priestley et al. (2007).
envelope as proposed by Paulay and Priestley (1992) is considered until a possible minimum flexural strength \( R_{\text{min}} \) is reached due to nominal minimum reinforcement requirements. According to Bachmann and Linde, \( \alpha_{\infty} = 0.20T_1 \) would be adequate only within a limited period range of about 0.5 to 2.5 s. The practical application of this design envelope is questionable because the flexural strength of a wall along its height normally decreases from the base to the top with the applied axial compression due to gravity loads. Consequently, it may be difficult to provide sufficient flexural reinforcement to generate the required increased strength \( R_e \) because of code-specified maximum reinforcement limits or construction issues resulting from reinforcement congestion.

### 2.1.5. Priestley and Amaris (2003)

For walls designed according to the direct displacement-based design (DDBD) method proposed by Priestley and Kovalsky (2000), Priestley and Amaris (2003) proposed a capacity design moment envelope that accounts for higher mode responses. This envelope is based on a modified modal superposition (MMS) approach that is an extension of the modal limit forces method proposed by Keintzel (1990) for predicting the base shear demand on cantilever walls. This approach recognizes that ductility at the wall base primarily acts to limit first mode response, but has comparatively little effect in modifying the elastic response in higher modes. Consequently, the elastic contribution of higher mode responses produces a flexural demand increase at the upper storeys as ground motion intensity increases. To account for that, Priestley and Amaris proposed that the capacity design moment at level \( i \) over the top half of the wall be determined with the MMS approach using the following relation:

\[
M_{\text{MMS,}i} = 1.1 \times \sqrt{M_{1,1}^2 + M_{2,1}^2 + M_{3,1}^2 + \ldots}
\]

with \( M_{1,1} = \min(M_{2,1}; M_{1,2}; M_{2,2}; \ldots) \) where \( M_{1,1} \) is the ductile design (first mode) moment at level \( i \) determined from DDBD and \( M_{1,2}, M_{2,2}, M_{3,2}, \ldots \) etc are the elastic modal moments at level \( i \) for lateral modes 1, 2, 3, etc. As the base moment is anchored to the flexural capacity of the wall, the profile of the capacity design envelope is considered linear from the mid-height moment to the overstrength moment capacity at the wall base, \( M_b^o \), as shown in Fig. 1. \( M_b^o \) is equal to \( \phi\delta M_{\text{base}} \) where \( \phi\delta \) is the flexural overstrength factor, defined as the ratio of overstrength moment capacity to required capacity of the base plastic hinge, and may be taken as 1.0 or 1.2, depending if steel strain-hardening is included or not in determining the required base flexural reinforcement, respectively. Priestley and Amaris pointed out that the MMS envelope tends in general to be slightly unconservative for short-period walls and rather conservative for long-period walls, for design-level ground motions.

### 2.1.6. Priestley et al. (2007)

Priestley et al. (2007) proposed a simplified version of the MMS envelope to avoid carrying out a modal analysis. As illustrated in Fig. 1f, this version consists in a bilinear capacity envelope defined by \( M_{b,1}^o \), the mid-height overstrength moment \( M_{b,1}^{0.5H} = A_T M_{b,1}^o \), and zero moment at the wall top, with the moment ratio \( A_T \) given by

\[ A_T = 0.4 + 0.075T_1 \left( \frac{\mu_\Delta}{\phi} - 1 \right) \geq 0.4 \]

where \( \mu_\Delta \) is the design displacement ductility ratio. Priestley et al. state that tension shift effects should be considered when curtailing flexural reinforcement. To that end, the capacity envelope should be shifted upwards assuming a tension shift equal to \( 1/2 \), as illustrated in Fig. 1f. It is interesting to note that eq. [2] is not bounded by an upper limit, meaning that the moment ratio could be equal to or larger than 1, as shown in Fig. 2 (with \( T = T_1 \)), resulting in \( M_b^o > M_{b,1}^o \). As previously discussed for the design envelope shown in Fig. 1d, the feasibility of such design may be simply impossible.

### 2.2. Shear strength design

The capacity design methods presented in this section aim to prevent shear failure over the entire height of a wall by providing a capacity design shear envelope corresponding to the development of the maximum feasible bending strength of the base plastic hinge and accounting for higher mode amplification effects through a dynamic shear amplification factor.

#### 2.2.1. CSA standard A23.3-04

As illustrated in Fig. 3a, CSA standard A23.3-04 requires that the capacity design shear envelope be greater of (i) the shear force corresponding to the development of the probable moment capacity, \( M_p \), of the wall base, which can be taken as recommended in the Explanatory notes on CSA standard A23.3-04 (CAC 2006), that is

\[ V_p = V_t \left( \frac{M_p}{M_t} \right)_{\text{base}} \]

and (ii) above the base hinge zone, the shear force, \( V_{ah} \), corresponding to the development of the factored moment resistance, \( M_t \), at the top of the base hinge zone \( h_p \), obtained as follows:

\[ V_{ah} = V_t \left( \frac{M_t}{M_t} \right)_{h_p,\text{top}} \]

but neither \( V_p \) nor \( V_{ah} \) shall be taken greater than the shear limit, \( V_{\text{limit}} \), determined from the elastic shear forces with \( R_d R_o = 1.3 \). Note that, for ductile walls (\( R_d R_o = 5.6 \), \( V_{\text{limit}} \) controls the shear strength design for walls with \( \gamma_s \geq 5.6/1.3 \approx 4.3 \). CSA standard A23.3-04 requires that the design envelope accounts for the dynamic amplification of shear forces due to inelastic effects of higher modes. However, no indication is given at this time to take into account this amplification. The new method proposed in this paper for shear strength design intends to address this deficiency. Also it considers a single envelope instead of two, \( V_p \) and \( V_{ah} \), while preserving an upper limit for walls considerably overstrengthened in flexure.

#### 2.2.2. Paulay and Priestley (1992)

The capacity design method of CSA standard A23.3-04 for shear strength design is based on that proposed by Paulay and Priestley (1992) where the capacity design shear envelope, \( V_{\text{E}}^o \), is obtained as follows (see Fig. 3b):

\[ V_{\text{E}}^o = \min \left( V_{\text{E}} \right) \]

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Priestley limit where

\[ N = \frac{V}{C_1} \]

is determined with the MMS approach

\[ \frac{M_{ud}}{M_e} \]

ratio of the wall base moment capacity, \( M_{ud} \), determined considering material overstrength and steel strain-hardening, to \( M_e \) at the wall base, and \( \omega_v \) is a dynamic shear amplification factor to account for higher mode amplification effects on shear forces, and is taken as

\[ \omega_v = \begin{cases} 0.9 + N/10 & \text{for } N \leq 6 \\ 1.3 + N/30 \leq 1.8 & \text{for } N > 6 \end{cases} \]

where \( N \) is the number of storeys of the building. Equation [6] is based on the work of Blakeley et al. (1975). Paulay and Priestley limit \( V_E^o \) to \( \muDelta V_E \), that is, the shear forces corresponding to the elastic response of the building. Various works (Keintzel 1990; Priestley and Amaris 2003; Rutenberg and Nsieri 2006) showed that eq. [5] can be very unconservative because of the significant underestimation of the higher mode shear amplification by \( \omega_v \).

2.2.3. Priestley and Amaris (2003)

Priestley and Amaris (2003) proposed that the capacity design shear envelope be determined with the MMS approach using the following relation:

\[ V_{MMS,i} = \sqrt{V_{i,1}^2 + V_{i,2}^2 + V_{i,3}^2 + \ldots} \]

with \( V_{i,j} = \min(V_{E,j}; V_{c,j}) \) where \( V_{E,j} \) is the ductile first mode shear force at level \( i \) determined from DDBD and \( V_{c,j} \) etc are the elastic modal shear forces at level \( i \) for lateral modes 1, 2, 3, etc. Priestley and Amaris reported that the MMS envelope is generally a little unconservative for short-period walls and slightly conservative for long-period walls, for design-level ground motions.

2.2.4. Priestley et al. (2007)

Alternatively to the MMS envelope, Priestley et al. (2007) proposed for a DDBD a simple capacity design shear envelope defined by a straight line between the base and the top of the wall, as shown in Fig. 3c. The capacity design base shear force, \( V_0^o \), is equal to \( \omega_v^o \phi_0 V_{Fr} \) with

\[ \omega_v^o = 1 + B_T \muDelta/\phi_0 \]

and

\[ B_T = 0.067 + 0.4(T_1 - 0.5) \leq 1.15 \]

where \( \omega_v^o \) is a dynamic shear amplification factor and \( \phi_0 \) is taken as \( M_{ud}/M_e \) at the wall base. The design shear force at the top of the wall, \( V_0^o \), is equal to \( C_T V_0^o \) with

\[ C_T = 0.9 - 0.3T_1 \geq 0.3 \]

2.2.5. 2004 Eurocode 8

As illustrated in Fig. 3d, the 2004 EC8 requires for ductile walls part of wall systems that the capacity design shear envelope, \( V_{Ed} \), be determined by amplifying \( V_f \) by \( \varepsilon \), a dynamic shear amplification factor, taken as 1.5 for moderately ductile (MD) walls \( (q \leq 3) \) and, for highly ductile (HD) walls \( (q > 3) \), calculated from eq. [11], which is based on the formula proposed by Keintzel (1990) to amplify the seismic shear forces obtained from the code-specified equivalent static analysis

\[ \varepsilon = q \sqrt{\left( \frac{Y_{Ed}}{q} \right)^2 + 0.1 \left( \frac{S_e(T_b)}{S_e(T_1)} \right)^2} \]

with \( 1.5 \leq \varepsilon \leq q \) where \( q \) is the behavior factor (equivalent to \( \muDelta \)) used for design, \( M_{Ed} \) is the design flexural resistance at the wall base, \( M_{Ed} \) is the design bending moment (equivalent to \( M_f \)) at the wall base, \( Y_{Ed} \) is the factor to account for overstrength due to steel strain-hardening (may be taken as 1.2), \( T_1 \) is the upper limit period of the constant spectral acceleration region of the spectrum and \( S_e(T) \) is the ordinate of the elastic acceleration response spectrum at period \( T \). For ductile walls part of frame-wall systems, the 2004 EC8 specifies that \( V_{Ed} \) be determined as shown in Fig. 3e where \( V_{db} \) is the capacity design shear force at the wall base. Note that the design envelope shown in Fig. 3d may have a similar profile to that shown in Fig. 3a if \( V_f \) is determined from dynamic analysis. Various works (Priestley and Amaris 2003; Rutenberg and Nsieri 2006) showed that eq. [11] tends generally to be conservative for short-period walls but unconservative for long-period walls, and that \( \varepsilon = 1.5 \) is increasingly unconservative for MD walls as \( T_1 \) and \( q \) increase.

2.2.6. Rutenberg and Nsieri (2006)

In an attempt to fix the underestimation issues of the 2004 EC8 method, Rutenberg and Nsieri (2006) proposed for isolated walls or walls part of uncoupled wall systems with similar wall lengths the capacity design shear envelope shown in Fig. 3f where the capacity design shear force at the wall base, \( V_{db} \), is taken as

\[ V_{db} = \varepsilon^* V_{by} = [0.75 + 0.22(T_1 + q + T_1 q)] V_{by} \]

\[ V_{by} = \frac{M_{by}}{(2/3)H[1 + (1/2N)]} \]

where \( \varepsilon^* \) is a dynamic shear amplification factor, \( V_{by} \) is the shear force corresponding to the flexural yielding at the wall.
base, $M_{by}$, under an inverted triangular force distribution over the height $H$ of a $N$ storey wall, and $\xi$ is taken as

$$\xi = 1.0 - 0.3T_1 \geq 0.5$$

Each one of the expressions $0.1H$ and $\xi H$ in Fig. 3f should be taken as an integer number of storeys. Equation [12] is based on the observation that dynamic amplification of base shear force increases quite linearly with $T_1$ and $q$. Rutenberg and Nsieri pointed out that the proposed envelope is fitted to $q = 1.0$, meaning that it is conservative for larger $q$ values. They added that eq. [14] is also applicable to cases where additional hinges develop at the upper storeys since shear amplification along the wall height decreases in such cases.

Recently Celep (2008) proposed a capacity design shear envelope for the Turkish seismic design code similar to that proposed by Rutenberg and Nsieri, with the following notable differences: $\xi = 0.4$ for any $T_1$, the base hinge height is the max($H/6$) and the capacity design base shear force is equal to $\beta^b V_f$ with the dynamic shear amplification factor $\beta^b$ given by

$$\beta^b = 1.0 + (0.281T_1 + 0.394)[(R/\psi^o) - 1.5]^{0.553}$$

with $1 \leq \beta^b \leq R$ where $R$ is the force reduction factor and $\psi^o$ is a flexural overstrength factor calculated as the ratio $M_f/M_t$ at the wall base.

### 2.3. Summary

From all methods outlined, those that are appealing and appear adequate for capacity design of ductile walls whose seismic force response is governed by higher mode responses is the bilinear moment envelope proposed by Priestley et al. (2007) for flexural strength design and the shear envelope proposed by Rutenberg and Nsieri (2006) for shear strength design. The simplicity of these two methods is appealing for practice. Moreover, the force envelope profiles proposed by these methods are in line with those predicted in the companion paper. In addition, both methods explicitly account for the influence of $T_1$ and the wall base bending strength, two parameters affecting significantly higher mode amplification effects, on the force envelope profiles. Their current form, however, is not adapted to Canadian codes and does not necessarily reflect the results presented in the companion paper.

### 3. Proposed capacity design methods

In the following are presented the new capacity design methods proposed for CSA standard A23.3 for flexural and shear strength design of regular ductile RC cantilever walls. The derivation of the proposed methods is based on the outcomes from the previous literature review and the parametric study presented in the companion paper. It is recalled that this study is based on two-dimensional inelastic time-history
analyses of fixed-base isolated RC cantilever wall models designed with the 2010 NBCC and CSA standard A23.3-04 and subjected to statistically independent simulated ground motion records compatible with the design spectra (2500 year return period) of different soil conditions of the seismic zone of Vancouver, which has the highest urban seismic risk in Canada. Phenomena that can significantly reduce the seismic forces resisted by a wall, such as foundation rocking, soil flexibility, and strength contribution coming from structural elements not part of the SFRS, were not taken into account. Therefore, the modeling considered for the parametric study represents an upper-bound case for a SFRS acting as a cantilever wall, and so are the predictions resulting from this modeling for the considered seismic zone. This conservatism in the predictions is accounted for in what follows. Note that the predictions presented in the companion paper are mean predictions obtained from 40 records.

3.1. Flexural strength design

For ductile walls, CSA standard A23.3-04 requires first that the critical section of the plastic hinge at the wall base be designed such that \( M_t \geq M_f \) and then, for capacity design considerations, that the wall sections above the base hinge zone \( h_t \) be designed such that \( M_t \geq M_f \) amplified by the ratio \( M_t/M_f \) calculated at the top of the hinge zone, as shown in Fig. 1a. The companion paper showed that this capacity design method can produce unconservative design envelopes for walls with \( T_1 \geq 1.0 \text{ s} \) and \( \gamma_w < 3.0 \). However, it was shown also that this method can generally limit the unintended plastic action above the base hinge zone to an acceptable level when \( \gamma_w \geq 2.0 \) and special curtailment considerations are applied. This suggests that a simple incorporation of these criteria to the CSA standard A23.3-04 method could enable to prevent plastic hinge formation above the base hinge zone, as desired. This option is not selected for three reasons. First, the enhanced CSA standard A23.3-04 method could still generate unconservative design envelopes at the upper storeys for walls with \( T_1 \geq 1.0 \text{ s} \) and \( 2.0 \leq \gamma_w < 3.0 \). Second, the approach of designing with an increased bending moment based on \( M_f \) for sections above the base hinge zone cannot capture the increase due to the inelastic action of the relative higher mode contribution in the flexural demand at the upper storeys because \( M_f \) is determined from a linear elastic analysis. Finally, this approach is not as appealing for practice as a simple design envelope as that shown in Fig. 1f.

The selected capacity design method is based on that of Priestley et al. (2007) for DDBD, which proposed the simple bilinear envelope shown in Fig. 1f. This design envelope requires determining only two parameters once the required flexural reinforcement at the wall base has been set: the overstrength moment capacity at the wall base, \( M_{o/b}^w \), and the moment ratio \( A_f \) (eq. [2]) of mid-height overstrength moment, \( M_{o/5H}^w \), to \( M_{o/b}^w \). Note that \( M_{o/b}^w \) is calculated as a nominal strength, that is, with characteristic lower-bound values for material strengths, while accounting for steel strain-hardening because it is determined at the curvature corresponding to the selected design displacement. This flexural strength is similar to the probable strength defined in CSA standard A23.3-04, though slightly greater because the probable strength is calculated with lower material strengths. Both parameters defining the bilinear envelope are modified in what follows based on the CSA standard A23.3-04 seismic design provisions and the results presented in the companion paper.

The CSA standard A23.3-04 specifies that the required flexural reinforcement of any section of a wall be determined such that \( M \geq M_f \). Based on the companion paper, it appears that imposing a minimum base overstrength \( \gamma_w^\min = 2.0 \) can generally limit the unintended plastic action above the base hinge zone to an acceptable level as far as special curtailment considerations are applied. It is proposed then that the design flexural strength requirement for the critical section of the base plastic hinge of regular ductile cantilever walls with \( T_1 > 0.5 \text{ s} \) be expressed as

\[
M_t \geq M_{o/b} = \phi_s \gamma_w^{\min} M_f = \gamma_s M_f
\]

where \( M_{o/b} \) is the minimum base overstrength moment, \( \phi_s \) is the steel resistance factor, taken as 0.85, as specified in CSA standard A23.3-04, and \( \gamma_w \) is the minimum factored base overstrength and is equal to 1.7. Note that \( \gamma_w = 2.0 \) was determined from inelastic time-history analyses of wall models for which the material strengths were the specified values used for design and the strain hardening and the Bauschinger effect of reinforcement steel were accounted for. Since the ratio of actual to specified material strength is generally larger than 1.0, it can conservatively be assumed that this ratio for yield strength of reinforcing bar steel in Canada is 1.05 (Mitchell et al. 2003). If this excess strength is accounted for, \( \gamma_w \) would be equal to 1.7/1.05 \( \approx 1.6 \). Although the requirement given by eq. [16] appears at first sight uneconomical from an engineering point of view, excess flexural strength in RC shear walls due to the required minimum reinforcement is common because the wall dimensions are more often governed by functional and architectural considerations than by seismic considerations. Moreover, preventing a possible plastic hinge formation at the upper storeys enables to save on the required special ductile detailing for an additional hinging region.

The moment ratio, \( \alpha_M \), of mid-height to base moment of the bilinear envelope is determined considering the minimum overstrength requirement proposed for the wall base and using the results presented in the companion paper. From this paper, moment ratios of the predicted mean moment demand at the wall mid-height to the nominal moment resistance at the wall base can be derived for \( T_1 \) values ranging from 0.5 s and 4.0 s and \( \gamma_w \) values equal to 2.0, 3.0, and 4.0. Figure 4 shows the maximum moment ratio for each selected \( T_1 \) value for the three \( \gamma_w \) values. Since a linear envelope as shown in Figs. 1b and 1c corresponds to a moment ratio of 0.50, lower moment ratio values are irrelevant for design purposes. Figure 4 indicates that a simple linear envelope is adequate for walls with \( \gamma_w \geq 4.0 \) irrespective of \( T_1 \). Moreover, this figure shows that, for walls with \( T_1 \geq 1.0 \), constant moment ratios of 0.62 and 0.55 are conservative for \( \gamma_w \) values of 2.0 and 3.0, respectively. Based on these results, Table 1 gives the proposed \( \alpha_M \) values for determining the mid-height moment, \( M_{o/5H}^w \), of the bilinear envelope. In Table 1, the effective force reduction factor \( R_d \gamma_w \) is used instead of solely \( \gamma_w \) to generalize the proposed \( \alpha_M \) values to cantilever walls designed for a ductility level different from that on which are based the \( \alpha_M \) values, that is, \( R_d \gamma_w = 5.6 \).

\[
\alpha_M = \frac{M_{o/b}^w}{M_f} = \frac{\gamma_w \phi_s \gamma_w^{\min} M_f}{M_f} = \gamma_w \phi_s \gamma_w^{\min} = \gamma_w^{\max}
\]
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Proposed that the capacity design base moment, linear envelope to account for tension shift. Therefore, it is tional conservatism coming from the vertical shift of the bi-termined for a nominal strength based on the specified mate-

For tension over its shear span (distance from maximum to zero bending moment), a tension shift of 0.5 shear over its shear span (distance from maximum to zero

to resist the entire shear, unless the expected plastic deforma-

tion in the fact that the common relation is based on plasti-

As shown in Fig. 1f, Priestley et al. (2007) proposed a vertical shift of 0.5lw of the whole bilinear envelope to account for tension shift. Based on a member subjected to constant shear over its shear span (distance from maximum to zero bending moment), a tension shift of 0.5lw corresponds to the case where the entire shear is resisted solely by shear rein-

Note that the moment ratio values shown in Fig. 4 would be lower or higher by about 20% if they were determined using the probable or factored moment resistance at the wall base, respectively. In addition, the results presented in the companion paper showed that the mean base moments predicted for all studied wall cases were never greater than 10% of the nominal moment resistance of the wall base. This means that basing the capacity design base moment on the probable strength while using the proposed $\alpha_m$ values given in Table 1 will add more conservatism to design. Further conservatism, however, appears unnecessary considering the conservatism already included in the proposed $\alpha_m$ values and the additional conservatism coming from the vertical shift of the bi-linear envelope to account for tension shift. Therefore, it is proposed that the capacity design base moment, $M_{cap}$, be determined for a nominal strength based on the specified material strengths, as specified by CSA standard A23.3-04.

As shown in Fig. 1f, Priestley et al. (2007) proposed a vertical shift of 0.5lw of the whole bilinear envelope to account for tension shift. Based on a member subjected to constant shear over its shear span (distance from maximum to zero bending moment), a tension shift of 0.5lw corresponds to the case where the entire shear is resisted solely by shear reinforcement, neglecting concrete contribution, and a shift of lw, as proposed by Paulay and Priestley (1992), to the case where the entire shear is resisted solely by concrete, neglecting shear reinforcement contribution. Such loading condition, however, is not representative of that of walls where higher mode responses constantly change in time the shear force profile along the height of the wall, and hence the height from the base of the resultant lateral force. In the absence of anything better, a shift of 0.5lw is likely reasonable since CSA standard A23.3-04 requires for ductile walls that the shear reinforcement in the plastic hinge region be designed to resist the entire shear, unless the expected plastic deformation is low.

At the wall base, however, the curtailment of flexural reinforcement cannot only account for the tension shift. It has to take into account the whole expected height, from the base, of plasticity, referred to as $h_p$. CSA standard A23.3-04 estimates this height as $0.5lw + 0.1H$, height over which special detailing for ductility is required. The companion paper showed that this relation is too conservative for tall walls with large flexural overstrength at the wall base. Actually it was observed that $h_p$ reduces, with respect to $H$, with increasing $H$ and $\gamma_w$, as shown in Fig. 5a from mean $h_p$ predictions normalized to $H$. The good correlations (correlation coefficients $r \approx 1$) and the exponent values close to $-1$ of the trend lines in this figure suggest that the relationship between $h_p$ and $H$ is almost linear, as observed in Fig. 5b. From the latter figure, two observations can be made: (i) the variability of the predictions is larger for low $\gamma_w$ values and large $H$ values, and (ii) the linear trend lines cross the ordinate axis at almost the same point. The second observation suggests that the constant term of the linear trend lines is independent of $H$ and $lw$, and depends only on a geometric parameter that was kept constant throughout the different wall cases studied in the companion paper: the storey height $h_s$. A $h_s$ value of 3.5 m was used, except for few cases where $h_s$ was 3.0 m. Based on Fig. 5b, the constant terms of the linear trend lines are lower than $h_s$. To account for the large variability in the $h_p$ predictions, the mean ($M$) plus one standard deviation (SD) predictions are considered and shown in Fig. 5c. From these predictions, the following relation for estimating $h_p$ is proposed for design purposes:

\[ h_p = 0.8h_s + \beta_H \geq \max(h_s, 0.5lw) \]

where $\beta_H$ is equal to 0.10, 0.05, and 0.03 for $R_dR_d/\gamma_w$ values of 2.8, 1.87, and 1.4, respectively. The lower bound of $h_p$ aims to ensure a minimum height of special ductile detailing, at least over the first storey from the base, while accounting for tension shift. The major difference of eq. [17] with the relations that can be found in the literature is that it recognizes the influence of the base flexural overstrength on $h_p$. Moreover, the form of eq. [17] slightly differs from that of the common relation for walls, that is, $h_p = \alpha h_w + \beta H$ where $\alpha$ and $\beta$ are constants. This difference may found an explana-

tion in the fact that the common relation is based on plastici-

ty lengths that were measured from RC beam tests and tests of RC walls laterally loaded at their top (Bohl and Adde-

bar 2011). Such tests do not adequately represent the seismic loading conditions on multistorey walls, especially tall ones. Note that, for one-storey ductile walls laterally loaded at their top by seismic forces, eq. [17] requires a special ductile detail-

ting over the entire wall height. This is too conservative. The code-specified relation $h_p = 0.5lw + 0.1H$, which would be an upper bound for such walls (Bohl and Addebar 2011), is more appropriate for this particular case, though the relation $0.5lw + \beta_H$ accounting for the base flexural overstrength might be a better alternative. This should be investigated be-

Table 1. Proposed $\alpha_m$ values for determining the mid-height moment $M_{0.5lh}$ of the bilinear envelope.

<table>
<thead>
<tr>
<th>$R_dR_d/\gamma_w$</th>
<th>$T_1 \leq 0.5$</th>
<th>$T_1 \geq 1.0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.80</td>
<td>0.50</td>
<td>0.62</td>
</tr>
<tr>
<td>1.87</td>
<td>0.50</td>
<td>0.55</td>
</tr>
<tr>
<td>$\leq 1.40$</td>
<td>0.50</td>
<td>0.50</td>
</tr>
</tbody>
</table>

From all the parameters previously set, a capacity design
moment envelope can be formulated. The proposed envelope for flexural strength design of regular ductile RC cantilever walls is illustrated in Fig. 6a. This envelope is obtained as follows:

1. Determine the minimum base overstrength moment $M_{b,\text{bo}}$ by scaling up the factored design base moment $M_{f,\text{base}}$ with the minimum factored base overstrength $g_b = 1.7$;
2. Determine the required flexural reinforcement content at the wall base to satisfy both eq. [16] and the minimum reinforcement requirements of CSA standard A23.3-04;
3. From this reinforcement, determine from sectional analysis the nominal base moment capacity $M_{nb}$ using the specified material strengths;
4. Calculate the wall overstrength factor $\gamma_w = M_{eb}/M_{f,\text{base}}$, and then $R_dR/f_{y,w}$ and the fundamental lateral period $T_1$ of the wall system, determine from Table 1 the moment ratio $\alpha_M$ and then calculate the mid-height moment $M_{0.5H} = \alpha_M M_{nb}$. Linear interpolation on $R_dR/f_{y,w}$ and $T_1$ may be used to get $\alpha_M$;
5. From $M_{eb}$ and $M_{0.5H}$, draw the bilinear envelope as illustrated in Fig. 6a;
6. Determine the plastic hinge height $h_p$ using eq. [17]. This height should be taken as an integer number of storeys. Linear interpolation on $R_dR/f_{y,w}$ may be used;
7. Vertically shift first the whole bilinear envelope of 0.5$V_{p,\text{bo}}$ and then the base vertical line up to $h_p$.
8. From $M_{eb}$ and $M_{0.5H}$, draw the bilinear envelope as illustrated in Fig. 6a;
9. The required flexural reinforcement at the wall base is maintained over $h_p$. For the wall sections above $h_p$, the required flexural reinforcement is determined by at least matching the nominal moment resistance of the section to the...
capacity design envelope. Bars to be curtailed must be extended a development length above the design envelop. Based on the companion paper, reinforcement curtailment should not result in a wall flexural strength reduction between two adjacent storeys that exceeds about 20% to 10% for walls with $T_1$ ranging from 1.0 s to 4.0 s, respectively.

### 3.2. Shear strength design

As mentioned in section 2.2.1., CSA standard A23.3-04 requires that the capacity design shear envelope accounts for the dynamic amplification of shear forces due to inelastic effects of higher modes. However, no method is specified to take into account this amplification. The new method proposed herein intends to address this deficiency. Also it proposes a single envelope instead of two, $V_p$ (eq. [3]) and $V_{db}$ (eq. [4]), while preserving an upper limit for walls with considerable flexural overstrength at their base.

The proposed capacity design shear envelope is based on that of Rutenberg and Nsieri (2006), which is illustrated in Fig. 3f. As shown in this figure, four parameters define the design envelope of Rutenberg and Nsieri: the capacity design base shear force $V_{db}$ (eq. [12]), which accounts for the higher mode shear amplification from a dynamic shear amplification factor, the capacity design top shear force, taken as $0.5V_{db}$, the height ratio $\xi$ (eq. [14]), and the base hinge height, taken as $0.1H$.

The design base shear force $V_{db}$ is replaced by the amplified probable base shear force $V_{pb}$, which is calculated as follows:

$$V_{pb} = \bar{\sigma}_v V_{pbase} \leq V_{limitbase}$$

where $\bar{\sigma}_v$ is a dynamic shear amplification factor, $V_{pbase}$ is the probable base shear force $V_p$ (eq. [3]) at the wall base, and $V_{limitbase}$ 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. The derivation of $\bar{\sigma}_v$ is based on the dynamic shear amplification results presented in the companion paper. These results are summarized in Fig. 7, which shows, for a given wall overstress factor value, the maximum dynamic shear amplification for each fundamental lateral period ($T_1$) value considered for the wall cases studied in the companion paper. Dynamic shear amplification is expressed as the ratio of the predicted mean wall shear force to $V_p$ at a given storey, the wall base in this case, or as an average of the ratios of all storeys (AOS). The predictions account for inelastic shear–flexure–axial deformation and interaction. Figure 7 shows that the dynamic shear amplification values largely increase with increasing $T_1$ from 0.5 s to 1.0 s and, for $T_1 \geq 1.0$ s, slightly increase for $\gamma_w \leq 2.0$ and remains almost constant for $\gamma_w \geq 3.0$, as $T_1$ increases. Also it shows that the base values are always greater than or equal to the AOS values.

Before proposing any $\bar{\sigma}_v$ values for design purposes, the following discussion needs to be addressed. To be conservative, it would be reasonable to set the proposed $\bar{\sigma}_v$ values larger than the base predictions shown in Fig. 7. Such conservatism, however, is judged unnecessary because of the following reasons. First, a certain conservatism level is already included in the predictions because of the conservative modeling used for analysis and of the high 2500 year return period of the design earthquake. Moreover, the parametric study performed in the companion paper did not predict any shear failure despite sometimes predicted peak base shear forces 3 times larger than the design shear resistance for walls with a $\gamma_w$ value of only 1.3. In addition, recent dynamic tests of large-scale 8-storey moderately ductile (MD) RC wall specimens designed according to CSA standard A23.3-04 and presenting light flexural overstrength at their base showed stable hysteretic shear responses, no shear reinforcement yielding, and no shear failure of the specimens for base shear demands corresponding up to 150% of the nominal shear resistance (based on the actual material strengths) of the wall or 200% of the design earthquake (Ghorbaniremani 2010). As discussed in the companion paper, the absence of shear failure for such high shear forces can likely be explained by a combination of the transient nature of these higher-modes dominated forces, where the associated energy is insufficient to sustain the displacement necessary for a shear failure, and of the high conservatism in the shear resistance requirements of CSA standard A23.3-04 for ductile and MD shear walls. The performance of recently built RC walls in Canada under future major seismic events might support these assumptions. Note, however, that the recent 2010 M 8.8 Chilean earthquake showed that RC walls designed according to modern design codes are not free from shear failure (Lagos 2011).

Based on the above observations, it seems that the potential risk of shear failure for ductile and even MD walls designed according to CSA standard A23.3-04 and whose seismic force response to design earthquake is dominated by higher mode responses is very low. From this remark, the use of $\bar{\sigma}_v$ may be queried, especially for low-period walls and walls with large base overstrength as dynamic shear amplification in the inelastic regime is low, as shown in Fig. 7. The $\bar{\sigma}_v$ values need not be taken larger than 1.0 for ductile walls with $\gamma_w \geq 4.0$ since the shear strength design of such walls is controlled by $V_{limit}$. Since life safety is a priority, Table 2 gives the proposed $\bar{\sigma}_v$ values for design purposes.

Based on the predicted wall shear force demands presented in the companion paper, the capacity design top shear force of $0.5V_{db}$ proposed by Rutenberg and Nsieri (2006) is ad-

![Fig. 7. Predicted maximum dynamic shear amplifications at the wall base and over all storeys (AOS) vs. fundamental lateral period for wall overstrength factor ($\gamma_w$) values equal to 1.3, 2.0, 3.0, and 4.0 (data from Boivin and Paulitre 2012).](image-url)
Table 2. Proposed $\bar{\omega}_v$ values.

<table>
<thead>
<tr>
<th>$R_dR_f\gamma_w$</th>
<th>$T_1 \leq 0.5$</th>
<th>$T_1 \geq 1.0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.80</td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>1.87</td>
<td>1.0</td>
<td>1.5</td>
</tr>
<tr>
<td>$\leq 1.40$</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

equate and conservative for multistorey walls. Thus, this parameter is taken as $0.5V_{pb}$ for the proposed envelope. The height ratio $\xi$, given by eq. [14] and shown in Fig. 3f, is another parameter defining the Rutenberg and Nsieri’s envelope. The $\xi$ values are bounded between 0.5 and 1.0. These values bound very well the top half of the predicted wall shear force demands along the wall height presented in the companion paper. Although not shown herein, these demands show that, regardless of $\gamma_w$, the $\xi$ values of 1.0 and 0.5 are adequate for $T_1 = 0.5$ s and $T_1 \geq 1.0$ s, respectively. From these observations, a new height ratio $\tilde{\xi}$ is proposed:

$$[19] \quad 0.5 \leq \tilde{\xi} = 1.5 - T_1 \leq 1.0$$

As shown in Fig. 3f, there is a height of $0.1H$ over which the design base shear is kept constant. This height, which intends to represent the plastic hinge region, is replaced by $h_p$ given by eq. [17].

Figure 6b illustrates, with the new parameters previously set, the proposed capacity design shear envelope for shear strength design of regular ductile RC cantilever walls. This envelope is determined as follows once the flexural reinforcement content at the wall base has been set:

1. Calculate first the probable moment resistance $M_p$ at the wall base from sectional analysis and then the probable base shear force $V_p$ at the wall base using eq. [3];
2. Determine the dynamic shear amplification factor $\bar{\omega}_v$ from Table 2 using $R_dR_f\gamma_w$ and the fundamental lateral period $T_1$ of the wall system. Linear interpolation on $R_gR_f\gamma_w$ and $T_1$ may be used.
3. Calculate the capacity design base shear force $V_{pb}$ with eq. [18] and the capacity design top shear force as $0.5V_{pb}$;
4. Calculate the height $\tilde{\xi}H$ using eq. [19] for $\tilde{\xi}$. This height should be taken as an integer number of storeys;
5. Determine the plastic hinge height $h_p$ with eq. [17];
6. Draw the capacity design envelope as shown in Fig. 6b.

### 4. Discussion

Any method has its limitations and so have the capacity design methods proposed in the previous section. First of all, the proposed methods are based on numerical simulations to which are associated assumptions, simplifications, and hence uncertainties. Despite the uncertainties deriving from the structural modeling or from the nonlinear time integration method adopted in this work, the main uncertainties underlying the predictions used to derive the proposed methods are by far the input earthquake and the damping because actually they are the two main unknowns in seismic analysis. In an attempt to minimize the ground motion uncertainty, 40 statistically independent simulated earthquake records compatible with the design spectra and representative of the magnitude-distance scenarios dominating the design-level seismic hazard of the selected seismic region were used for each studied wall case. For damping, an initial stiffness-based Rayleigh damping model was used with a modal damping ratio of 2% of critical, which is a typical mean value for multistory RC wall buildings (CTBUH 2008; Gilles 2010), assigned to the first and last lateral modes of the analyzed wall to avoid possible problems of spurious damping forces, and hence of force equilibrium, resulting from this Rayleigh damping formulation (Crisp 1980). Although better damping models exist, the selection of this damping model was dictated by the use of the finite element program VecTor2 (Wong and Vecchio 2002) where only this damping model is implemented in the program. Nevertheless, Léger and Dussault (1992) recommended Rayleigh damping for seismic analysis of MDOF building structures with $T_1 > 0.5$ s and showed that the influence of the selected Rayleigh damping formulation is not so significant on the seismic response and becomes negligible for structures with $T_1 > 1.5$ s. Based on that, the utilized Rayleigh damping model should not have affected much the predictions. However, the significant variability, which is on the order of 30% to 40% (Porter et al. 2002; Gilles 2010), in modal damping ratios measured in actual multistory RC wall buildings was not taken into account. In spite of that, it appears reasonable to consider that the obtained predictions are in some manner conservative because the selected damping model has assigned modal damping ratios way below 2% of critical, especially for tall walls, to dominating higher lateral modes.

An important limitation of the proposed methods comes from the fact that the predictions are specific to the seismic region of Vancouver, which has a seismic hazard that is representative of that of western Canadian cities. For eastern Canadian cities, the proposed values of the parameters defining the design envelopes may be unconservative because the typical ground motions of the eastern regions are generally high-frequency motions rather than low-frequency motions, as those of the western regions. High-frequency motions excite further higher mode responses and hence may produce larger dynamic amplification effects. For instance, Boivin and Paultre (2010) studied the seismic performance of a ductile RC core wall structure designed according to the 2005 NBCC and the CSA standard A23.3-04 for the seismic zone of Montreal, an eastern Canadian city having the second highest urban seismic risk in Canada. The core wall consists of a cantilever wall system in one direction and a coupled wall system in the orthogonal direction. In the cantilever wall direction, $T_1 = 1.74$ s and $\gamma_w = 3.6$. Based on the predicted mean seismic force demands for design-level ground motions presented in their paper, the ratio of the predicted mid-height moment to the base moment resistance is about 0.6 and the dynamic base shear amplification, with respect to $V_{pb}$, is about 1.5 for the isolated cantilever wall system. From the previous $T_1$ and $\gamma_w$ values, Tables 1 and 2 give by linear interpolation a moment ratio $\alpha_M$ of about 0.52 and a dynamic shear amplification factor $\bar{\omega}_v$ of about 1.17, respectively. These values are lower than the previous ones. This suggests that the proposed $\alpha_M$ and $\bar{\omega}_v$ values can be unconservative for eastern regions having a seismic hazard similar to that of Montreal. Note, however, that the moment ratio value of 0.6 and the dynamic base shear amplification value of 1.5 taken from Boivin and Paultre (2010) were obtained from a wall model where shear deformation was considered
linearly elastic. Such wall model for inelastic simulations tends in general to overestimate force demand, especially shear demand, meaning that both previous considered values most likely overestimate the actual force demand. A work similar to that conducted in the companion paper is in progress to derive adequate capacity design envelopes for the eastern Canadian regions.

Since the proposed methods are based on predictions obtained from isolated regular RC cantilever wall models, their application for seismic design of SFRSs is essentially for RC cantilever walls that are regular and uniform in strength and stiffness over the whole height of the building and are part of a system acting as a single cantilever wall, such as a core wall in a tall building. This equivalence of lateral behavior is possible only if the cantilever walls constituting the system have similar cross sections and lengths and if the system is not irregular in torsion. For systems significantly outside these specifications, the validity of the proposed methods needs to be investigated. Actually, for systems constituted of cantilever walls with largely different cross sections and lengths, the large variations in stiffness and strength of the walls can produce in the inelastic regime shear distributions among the walls that are very different from those usually based on their relative stiffness or relative flexural strength because of system-related phenomena, such as the sequence of hinge formation (Rutenberg and Nsieri 2006) or the relative inelastic shear deformation (Adebar and Rad 2007) between the walls. The proposed methods also apply to cantilever walls that are part of RC wall-frame systems where the walls govern the lateral behavior of the system because in such systems dynamic amplifications are controlled and mainly resisted by the walls (Kabeyasawa et al. 1983). Thus, designing such walls with the proposed methods should produce conservative designs. For shear strength design, a less conservative approach would be to account for the relative participation of the walls in resisting shear in the entire wall-frame system. Considering the relative wall participation in such system, Paulay and Priestley (1992) proposed a relation based on \( \alpha_v \) (eq. [6]) to calculate a reduced dynamic shear amplification factor for estimating the wall base shear force for capacity design. By replacing \( \alpha_v \) by \( \bar{\alpha}_v \) (Table 2) in this relation, the following reduced amplification factor \( \bar{\alpha}_v \) may be used to calculate \( V_{pb} \) (eq. [18]) for walls that are part of wall-frame systems:

\[
\bar{\alpha}_v = 1 + (\bar{\alpha}_v - 1) \eta
\]

where \( \eta \) is the portion of the total base shear of the entire structure resisted by the walls.

Although the proposed capacity design methods were derived for ductile RC cantilever walls designed with CSA standard A23.3-04, their application can be extended to RC cantilever walls designed for any lower ductility level. For MD cantilever walls (\( R_dR_o/\gamma_w = 2.0 \times 1.4 = 2.8 \)), CSA standard A23.3-04 does not specify any capacity design provisions for their flexural strength design. Yet these walls are often used as SFRS for multistorey buildings, and hence are also prone to higher mode amplification effects. For instance, assuming a MD wall with \( T_1 = 1 \) s and \( \gamma_w = 1.3 \), \( R_dR_o/\gamma_w = 2.15 \) and Table 1 gives a \( \alpha_M \) value of about 0.57. This means that this wall with the minimum base flexural overstrength might experience an increased moment at the wall mid-height due to dynamic amplification effects. Note that the new minimum flexural overstrength requirement proposed for ductile walls (eq. [16]) does not apply to MD walls because the possible \( R_dR_o/\gamma_w \) values for MD walls will always be lower than 2.8. For shear strength design of MD walls, CSA standard A23.3-04 already specifies capacity design provisions, which are similar to those specified for ductile walls. These provisions could be superseded by the proposed capacity design shear method. Using the previous MD wall example, Table 2 gives a \( \bar{\alpha}_v \) value of about 1.65, which is an upper bound for MD walls. The excellent shear performance of the MD wall specimens dynamically tested by Ghorbanirenani (2010) for motion intensities corresponding up to 200% of the design earthquake suggests, however, that a \( \bar{\alpha}_v \) value of 1.0 would be sufficient for MD walls. This should be further investigated.

5. Conclusion

In this work, first a short review was conducted about the various capacity design methods proposed in the current literature and recommended by design codes for determining capacity design moment and shear envelopes for a SPH design of ductile RC cantilever walls. In this review, the reviewed methods were outlined as well as their limitations in estimating the seismic force demand on ductile walls whose seismic force response is governed by higher mode responses. Afterwards were presented the new capacity design methods proposed for CSA standard A23.3 for determining, for a SPH design, capacity design envelopes for flexural and shear strength design of regular ductile RC cantilever walls used as SFRS for multistory buildings. The derivation of these methods is based on the outcomes from the literature review and the parametric study presented in the companion paper. Finally a discussion on the limitations of these new methods and on their applicability to various wall systems was presented. This discussion highlighted the need of investigating on the two following issues:

1. The applicability of the proposed capacity design methods for systems constituted of cantilever walls with largely different cross sections and lengths.
2. The actual risk of shear failure of MD and ductile walls designed according to CSA standard A23.3-04 due to high peak shear forces generated by higher mode responses.

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